AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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410.

(Vol. XX.—April, 1889.)

# ENGLISH AND AMERICAN RAILROADS COMPARED IN OPERATING EXPENSES.

By EDWARD BATES DORSEY, M. Am. Soc. C. E.

For the information of a very prominent official and large shareholder in English railways, the author made some of the following comparisons. The result was so astonishing that he decided to enlarge upon it, and make a short paper, in hope of getting the subject freely discussed. The English and American practice differ so widely that both can not be right; engineers and railroad officials should discuss it freely, and adopt the principle that is the best and most economical, regardless of its nationality.

When the author commenced this investigation over five years ago, it was with the intention of showing the American engineers that it was the best policy to build good roads at first, as the English do, and save in the cost of operating expenses. The author then thought that it would be possible, on roads with large traffic, to save more than the additional interest charge, but as the investigation progressed, he was greatly astonished to find that the Americans were operating their roads much cheaper than the English, though built probably at less than one-fifth of the average cost of English railways.

This so surprised him that he thought there must be some mistake in his data and investigations. Acting upon this belief, he postponed

the reading of the first paper for six months, until he could again visit England and re-examine the subject and data.

It is now more than three years since his first paper entitled "English and American Railroads Compared," was read before this Society. Since then the subject has been much discussed by the press and railroad officials. About the only thing that has been said against it is, that it was based upon assumed data; in reality all the important figures given in it were from official sources, except the tonnage and passenger mileage; unfortunately the English roads do not give these. In reference to this, the author will quote from his first paper or book, page 16:* "No return of these all-important items for comparison is made in England. The author, after careful inquiry and investigation, decided that the average freight charge on all freight moved in the United Kingdom was about 1¼ pence, or 2.5 cents per mile per ton; but in order to be conservative he has taken the average charge at 1 penny, or 2 cents per ton per mile."

"In order to get the ton mileage, the receipts from freight, as reported by the companies, in pounds sterling has been multiplied by 240."

In the supplementary paper a year later, and after more thorough investigation had been made, the author said on page 790:†

"He hoped that the English railways which are so largely interested in this question will promptly replace his estimate of ton and passenger mileage by their official figures. Until this is done he claims that these figures should be accepted as correct."

Since this was written it has been investigated for two years more, fully confirming the preceding opinion and data.

All the data and figures in this paper are taken, for the English roads, from the "Board of Trade Railway Returns," or "General Report," and for the American roads from "Poors' Manual," or the annual reports of the companies.

In traffic expenses the Pennsylvania Railroad includes "rates and taxes," and "compensation for personal injury" and "compensation for damage and loss of goods," amounting to $322 370 for 1887, which should be deducted to make it correspond to the English roads, as on them they are not included in traffic expenses.

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* Transactions, Vol. XV, No. 318, January, 1886.
† Transactions, Vol. XVI, No. 343, October, 1886.

In the accompanying table the five principal English railways have been selected. Among the American railroads the Pennsylvania Railroad Division has been selected as representing the best constructed American road, and the Knoxville Branch of the Louisville and Nashville system as a sample of the cheaply constructed class, both running through a comparatively rough country. Both of these roads have most of their traffic in one direction. This reason is very frequently given by English officials for the high freight rates on their railways.

At first the author included in the Pennsylvania system, the Philadelphia and Erie, and the united railroads of New Jersey divisions, but on reflection, he thought best to exclude them, as it might be said that the level or easy grades on these roads were particularly favorable to cheap transportation. The Pennsylvania Railroad Division was retained, as it traverses a broken country with many heavy grades, and one summit of 2 154 feet above tide.

In absence of official data as to the average charges for transporting one ton of freight one mile on the English railways, the author has assumed that the average charge is one penny, or two cents, per ton per mile (one pound sterling = $4.80), being the same figures as given in his book and papers on pages 16 and 789, which figures have not been contradicted in any way, but have been repeatedly confirmed.

These comparisons speak very strongly in favor of the American system. The high terminal charges (including receiving and delivering freights by carts), are generally given as an excuse for the very high freight charges in England; the last two paragraphs show that there must be other reasons than these. In absence of any data as to what this expense amounts to, the author has in paragraph, as noted, deducted from the operating expenses of the English railways everything included under the name of "Traffic Expenses" (coaching and merchandise); this must certainly include all terminal charges and cartings. On the American railroads no deductions of any kind from the total operating expenses have been made; notwithstanding this, there is still a very large percentage in favor of the American roads, on the Pennsylvania it being 56 per cent. in its favor, compared with the London and North Western.*

*Being 144 per cent. in favor of the Pennsylvania Railroad when compared to the London and North Western.

In paragraph 11 all traffic expenses (coaching and merchandise) have been deducted from the operating expenses of all roads; this shows a very large percentage in favor of the American railroads.

Paragraph 6 shows the error in the statement usually made that the English railways are operated more economically than the American, on account of the operating expenses of the English roads being about 52 per cent. of the earnings, while the American roads are about ten per cent. higher. If the earnings on both roads were derived from the same charge for moving freight, the operating expenses of the Pennsylvania road would be only 21 per cent. of the earnings against 52 on the London and North Western, or 148 per cent. in favor of the American road.

It is frequently stated that the English railways are operated more cheaply than the American, because the train mile on the former costs somewhat less than it does on the latter, which is entirely wrong, if proper allowance is made for the difference in the average number of tons in the train load, which average, as per paragraph 3, is 79 tons on the London and North Western, against 207 on the Pennsylvania. Paragraph 13 shows that if proper equation is made for the heavier load, the cost per train mile will stand on the London and North Western $.6588, against $.3365 for equal load on the Pennsylvania, or 96 per cent. in favor of the latter.

There must be some cause for this great difference in the operating expenses of these roads; in the author's opinion the following are the principal reasons:

*First.*—Too light loads to the English trains, both freight and passenger.

*Second.*—The universal use on American roads of rolling stock with bogie-trucks, which move with much less friction and wear and tear than the rigid wheel base rolling stock used by the English railways.

*Third.*—The use on the American roads of freight cars carrying a much greater percentage of paying load to dead weight than the wagons used on the English railways.

*Fourth.*—The great speed at which the English freight trains run.

*Fifth.*—The use on the English roads of light locomotives, with inside connections that cannot be as cheaply or as easily repaired as the American locomotive, with its outside connections.

*Sixth.*—The custom on the English railways of giving each indi-

TABLE No.

## ENGLISH AND AMERICAN RAILROADS COM

Vol. XX, page 134.

1887.

£1 = $4.80. 1d. = 2 cents.

All figures representing cost are in cents, except the average cost per mile of railway operated, which is in dollars.

| | Pennsylvania Railroad Division. | Knoxville Branch, Louisville and Nashville System. | London and North Western. | Percentage against L. & N. W. and in favor | | Great Northern. | Percentage against Great Northern and in favor | |
|---|---|---|---|---|---|---|---|---|
| | | | | Pennsylvania Division. | Knoxville Branch. | | Pennsylvania Division. | Knoxville Branch. |
| Average cost per mile of railway operated, in dollars | ..... | $39 190 | $255 888 | ..... | ..... | $195 500 | ..... | ..... |
| 1. Percentage of total tonnage mileage transported East | 74 | 30 | ..... | ..... | ..... | ..... | ..... | ..... |
| 2. Percentage of total tonnage mileage transported West | 26 | 70 | ..... | ..... | ..... | ..... | ..... | ..... |
| 3. Average tons in loads of freight trains | 207 | 154 | 79 | 162 | 96 | 60 | 245 | 157 |
| 4. Average charge for transporting one ton one mile, cents | .67 | 91 | 2.00 | 200 | 120 | 2.00 | 200 | 120 |
| 5. Percentage of operating expenses to earnings | 63 | 58 | 52 | ..... | ..... | 56 | ..... | ..... |
| 6. Percentage of operating expenses to earnings on the Pennsylvania Railroad Division, provided it charged the same rates for freight as are charged on the English railways, viz., 1d. = 2 cents per ton per mile | ..... | ..... | 21 | 148 | ..... | 21 | 148 | ..... |
| 7. Percentage of increased earnings that the Pennsylvania Railroad Division would receive if it worked with their present operating expense and received the same freight rates (1d. per ton per mile), as the English railways now receive | ..... | ..... | 545 | ..... | ..... | 545 | ..... | ..... |
| 8. Percentage of traffic expenses (coaching and merchandise) to total operating expenses | 35 | 32 | 36 | ..... | ..... | 34 | ..... | ..... |
| 9. Average cost of transporting one ton one mile. The cost for the English railways is found by multiplying the estimated charge of 1d. per ton per mile by the percentage of operating expenses to the gross earnings, as per paragraph 5 above; cents | .426 | .504 | 1.04 | 144 | 106 | 1.12 | 163 | 122 |
| 10. Average cost of transporting one ton one mile, deducting on the English railways all "Traffic Expenses (coaching and merchandise)," and deducting nothing from the total operating expenses of the American railroads; cents | .426 | .504 | .666 | 56 | 32 | .739 | 74 | 47 |
| 11. Average cost of transporting one ton one mile, deducting all "Traffic Expenses" (coaching and merchandise) on all roads; cents | .277 | .343 | .666 | 140 | 94 | .739 | 167 | 116 |
| 12. Average cost of train mile—freight and passenger; cents | 85.37 | 71.00 | 65.88 | ..... | ..... | 55.42 | ..... | ..... |
| 13. Equated cost of train mile on the English railways, provided they transported their present train loads at the same cost per ton per mile as is done by the American railroads, to which they are compared, in cents | ..... | ..... | Cost..... | 33.65 | 39.82 | Cost.... | 25.56 | 30.24 |
| | | | Per cent. | 96 | 65 | Per cent. | 117 | 83 |

## TABLE No. 50.

### AMERICAN RAILROADS COMPARED IN OPERATING EXPEN

| don and North Western. | | | Great Northern. | | | Midland. | | | Great Weste | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Percentage against L. & N. W. and in favor | | | Percentage against Great Northern and in favor | | | Percentage against Midland and in favor | | | Perce aga Great and in |
| | Pennsylvania Division. | Knoxville Branch. | | Pennsylvania Division. | Knoxville Branch. | | Pennsylvania Division. | Knoxville Branch. | | Pennsylvania Division. |
| 88 | ..... | ..... | $195 500 | ..... | ..... | $256 727 | ..... | ..... | $161 030 | ..... |
| | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | 162 | 96 | 60 | 245 | 157 | 61 | 240 | 152 | 64 | 223 |
| 0 | 200 | 120 | 2.00 | 200 | 120 | 2.00 | 200 | 120 | 2.00 | 200 |
| | ..... | ..... | 56 | ..... | ..... | 52 | ..... | ..... | 49 | ..... |
| | 148 | ..... | 21 | 148 | ..... | 21 | 148 | ..... | 21 | 148 |
| | ..... | ..... | 545 | ..... | ..... | 545 | ..... | ..... | 545 | ..... |
| | ..... | ..... | 34 | ..... | ..... | 34 | ..... | ..... | 31 | ..... |
| 4 | 144 | 106 | 1.12 | 163 | 122 | 1.04 | 144 | 106 | .98 | 130 |
| 66 | 56 | 32 | .739 | 74 | 47 | .686 | 60 | 36 | .676 | 59 |
| 66 | 140 | 94 | .739 | 167 | 116 | .686 | 148 | 100 | .676 | 144 |
| 88 | ..... | ..... | 55.42 | ..... | ..... | 55.40 | ..... | ..... | 58.10 | ..... |
| .... | 33.65 | 39.82 | Cost.... | 25.56 | 30.24 | Cost.... | 25.99 | 30.74 | Cost..... | 27.26 |
| nt. | 96 | 65 | Per cent. | 117 | 83 | Per cent | 113 | 80 | Per cent. | 113 |

## EXPENSES.

| reat Western. | | | Great Eastern. | | | United Kingdom. | | |
|---|---|---|---|---|---|---|---|---|
| | Percentage against Great Western and in favor | | | Percentage against Great Eastern and in favor | | | Percentage against all Railways of United Kingdom and in favor | |
| | Pennsylvania Division. | Knoxville Branch. | | Pennsylvania Division. | Knoxville Branch. | | Pennsylvania Division. | Knoxville Branch. |
| 30 | ..... | ..... | $212 895 | ..... | ..... | $207 408 | ..... | ..... |
| ... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| ... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | 223 | 140 | 62 | 234 | 149 | 70 | 196 | 120 |
| 0 | 200 | 120 | 2.00 | 200 | 120 | 2.00 | 200 | 120 |
| | ..... | ..... | 53 | ..... | ..... | 52 | ..... | ..... |
| | 148 | ..... | 21 | 148 | ..... | 21 | 148 | ..... |
| | ..... | ..... | 545 | ..... | ..... | 545 | ..... | ..... |
| | ..... | ..... | 31 | ..... | ..... | 31 | ..... | ..... |
| 8 | 130 | 94 | 1.16 | 149 | 110 | 1.04 | 144 | 106 |
| 76 | 59 | 34 | .731 | 72 | 45 | .718 | 69 | 43 |
| 76 | 144 | 97 | .731 | 164 | 113 | .718 | 160 | 109 |
| 0 | ..... | ..... | 56.04 | ..... | ..... | 60.52 | ..... | ..... |
| ... | 27.26 | 32.26 | Cost..... | 26.91 | 31.25 | Cost..... | 29.82 | 35.28 |
| ent. | 113 | 80 | Per cent. | 112 | 79 | Per cent. | 100 | 72 |

vidual shipper the exclusive use of a freight wagon, even for very small quantities of goods.

The Pennsylvania Railroad Division is one of the few American railroads that approximate in thoroughness of construction to the average English roads. Even on this line there are many cheap station buildings, platforms, etc., that require constant repairs and renewals. Consequently, their maintenance is more expensive than the permanent masonry structures on the English roads. This applies still more forcibly to the inferiorly constructed American roads.

Of the 150 000 miles of railroads in the United States, it is safe to say that there is not 5 per cent. of them ballasted up to the English average standard, and perhaps more than one-half of the remainder without any pretense whatever to ballast.

The large English railways are very similar to each other in the general details of their operations; this is shown by the preceding table.

From all the data that are available, it appears that the cost of transporting one mile, one ton or one passenger, is about the same on the principal English railways. The question has been frequently asked, "How can the English railways pay the interest on their great cost, unless they continue to charge their present high freight rates?" This is entirely foreign to the object of this paper, which proposes only to discuss the cost of operating expenses in the two countries. The author is, however, thoroughly convinced, that on the English railways large reductions could be made in the present freight rates and cost of operating expenses, without diminishing the present dividends in the least.

On the English railways there is a great variety of rolling stock, differing very widely; this is especially true regarding the locomotive. It is not possible that these different varieties, differing so very much, can all be equally good. It is strange that the best type has not been selected, and generally adopted, especially on the same road. During a recent visit to Stoke-on-Trent, the author was told by the Duke of Sutherland, in reply to the question why he did not work his coal and iron mines more extensively, as the situation was very favorable for producing cheap coal and iron, that the railroad freights were nearly prohibitory, being to Liverpool—55 miles—7 shillings = $1.75 per ton, or 3.2 cents per ton per mile; compare this with the charge on the Louisville and Nashville Railroad of $1.40 per ton from Birmingham

to Mobile, 280 miles, or half a cent per ton per mile, or one-sixth of the English charge, and this over a much inferiorly constructed road.

A prominent railway official gave the usual reason for this high freight rate, that it was owing to the high terminal charges at Liverpool. There were none at Stoke, the iron being loaded into the cars by the furnace company. No data are given by which these terminal charges can be ascertained, but to be certain that we get all of them, let us deduct from the operating expenses all the "Traffic Expenses" (including coaching and merchandise), 36 per cent., as in paragraphs 8 and 10 of the preceding table, this will leave $1.12 as the freight charge, or over two cents per ton per mile, after deducting all traffic expenses. Compare this with the rate on the Louisville and Nashville Railroad of half a cent per ton per mile, and without any deduction whatever from the total operating expenses.

The preceding percentages of the American railroads would be very largely increased, if proper allowance be made for the higher wages paid, their difference in first cost—fully one to five, and consequently inferior construction, and their perishable wooden buildings and bridges.

It is now five years since the author commenced this investigation. All the data collected in this time prove that on the American railroads the total cost of transporting one ton per mile is less than half of what it is on the English roads.

The author will conclude this paper with the concluding sentence of his first paper: "For what is done in the United States ought to be done in the United Kingdom."

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411.

(Vol. XX.—April, 1889.)

# STRESSES IN BRIDGES.

By WILLIAM H. BOOTH, M. Am. Soc. C. E.

It is not possible to read the various papers and discussions which appear in America upon specifications for and stresses in bridges without perceiving that bridges form a subject which receives a large amount of the attention of American Engineers. To the present writer it appears that the tendency of modern American and the best English practice is towards a reduction in the amount of the so-called factor of safety which it has now become fashionable to call the factor of ignorance, though it should more truthfully or euphoniously be termed the factor of uncertainty. This reduction is clearly to be made concurrently with a more accurate determination of actual as distinguished from apparent loading, and it is clear that should a final determination of the full effect of any and every load upon a structure be accurately made, the factor of uncertainty would be at once replaced by a marginal factor, which would serve to determine the true working strength of a structure under known load, and such working load would differ little from the loading that would cause final rupture. In fact, the present wrongly termed factor of safety might then be consistently so termed.

The difficulties standing in the way of such an improvement in practice are chiefly connected with our ignorance of the actual stresses caused by a moving load, for whilst static loads can be accurately measured, there is difficulty in truly measuring the effects of a dynamic load.

The old principles, which allowed a uniform stress upon every part of a bridge, without reference to its place in the structure, have been discarded, and modern methods fully recognize that different details require different working stresses, and parts subject to suddenly applied loads are to be calculated as for static loads of much greater magnitude.

In actual practice, a study of American literature in the many papers and pamphlets so kindly sent the writer by many American engineers, to whom he would here take the opportunity of recording his thanks for their great courtesy, shows that in bridge work the points presenting most uncertainty are the percentage of increased stress due to quickly applied loads, such as that of a rapidly moving train, and the excess of weight due to the horizontal balance of locomotives disturbing their vertical equilibrium. These two causes belong solely to the loading, and may be classed as primary.

The secondary causes of increased load action are those connected wholly with the structure. First of these may be named general deflection of the whole structure, which induces increased stress by centrifugal action. Next in order comes the deflection of parts of the floor system, such as the stringers, which may induce vibratory action by the repeated raising of the rolling loads as they cross the panel points—*i. e.*, at the floor beams.

The third and perhaps most severe causes of increased stress are those which depend upon certain mutual relations between the load and the structure, and foremost of these is the relation between the drive wheel circumference and the panel length of the bridge. Next is the relation between wheel spacing of the train and the panel length; and finally, there is the still more complex relation between panel length, drive wheel circumference or train wheel spacing, velocity of motion of the train, and period of vibration of the bridge.

When a tie bar is stated as being loaded to a working stress of 10 000 pounds, engineers understand that the static load is intended. They are aware, perhaps, that the material should bear any number of applications of a 20 000-pound stress, but dare not approach such a limit simply because they know that unknown stresses are brought about by the above enumerated causes, which would work destruction speedily to a structure nominally calculated for 20 000 pounds; for it is now generally conceded that the nominal 10 000 pounds is more nearly 15 000 to 20 000. It would therefore appear desirable that actual

live load stresses should be determined; for there can be no doubt that if fully recognized that live loads do daily cause stresses of 15 000 to 20 000 pounds, it would soon be the custom to allow for dead loads this amount in calculation and the same also for the real live load. The effect would be most striking in long spans, where dead load forms so great a proportion of total load, and marked economy would arise, whilst long spans would still be no less strong than short spans. Indeed it would appear that either existing long spans are wastefully strong or short spans doubtfully safe.

Without attempting any scientific determination of live load stresses the writer would point out what appears to him a correct or at least a rational method of procedure, to be followed in determining the total action of moving loads. This, it appears to him, ought to be expressed as equivalent static load when possible. Commencing with the primary causes of the necessity of an ignorance factor, it will be recognized that in calculating bridge stresses by the modern system of wheel concentrations as given by Burr in "Stresses in Bridge and Roof Trusses," the essence of the principle demands that such wheel concentrations be known accurately. To this end full account must be taken of the vertical action of the counterweights in the drive wheel, and when a bridge is to be constructed there must be assumed a maximum speed at which the engine representing the maximum loading shall cross the bridge. Knowing this, the amount of the counterweight vertically unbalanced must be found and its centrifugal action calculated from the formula $F = .00034\ WrN^2$; $W$ being in pounds, $r$ in feet, and $N$ being revolutions per minute at given maximum velocity. It will not usually be necessary to assume $N$ to be increased by slipping of drive wheel, as a bridge is rarely on a steep gradient. The amount of $F$, when wheels are small, will be found quite serious, and must be added to the apparent weight on the driving axle—*i. e.*, to each wheel on such axle—and prudential reasons would also demand that the vertical component of the maximum piston load due to the angularity of the connecting rod should also be added to the load on the drive wheel, though it may lessen the load upon the leaders or truck wheels.

Thus altered, the engine concentrations may be assumed with correctness to be the actual loads, and it now concerns us to know to what extent such actual wheel loads must be further augmented to allow for rapidity of application. Here again a maximum train speed must be assumed.

It is with considerable diffidence that the writer would suggest a new method of determining percentage co-efficients for different lengths of span; but the method he proposes has a rational basis.

If a single heavy load roll upon a bridge at a certain velocity, the bridge will commence to deflect until the load attains mid-span, when further deflection ceases.

Consequently, the whole deflection must take place in the time occupied by the load in traversing half the span. Assuming that a train headed by a heavy locomotive is in the position of a single load—an error in assumption on the safe side—we may say that in a bridge of 180 feet span a train at 60 miles per hour speed will put the bridge into its position of maximum deflection in one second of time, for the engine will then stand at mid-span. Now, in one second any free load would fall under the action of gravity through a space of 16 feet, for $t^2 \times 16$=space traversed.

Assuming a deflection of $\frac{1}{400}$ of the span, we find this to be $\frac{9}{20}$ foot. Falling under gravity; $\frac{9}{20}$ of a foot is passed through in $\frac{1}{6}$ of a second;

$$\left\{\text{for} \quad 16\ t^2 = \frac{9}{20} \cdot \cdot \cdot t^2 = \frac{9}{320} \cdot \cdot \cdot t = \frac{3}{18}\right\}$$

The load, however, occupies one second in getting to mid-span. It therefore occupies six times the period necessary to deflect the bridge to its full extent. Now, an absolutely instantaneous application of load is equivalent to the steady application of a load of twice the weight, and demands an increase of 100 per cent. upon its static weight. In the present case, then, time being extended sixfold, we find $\frac{100}{6} = 16\frac{1}{2}$ per cent. as the proper increase in case of spans of 180 feet under loads advancing 90 feet per second. Calculated on this basis, the annexed table will give the percentage increase for the various spans enumerated, speed being taken at 90 feet per second and deflection (assumed exaggerated) at $\frac{1}{400}$ of span.

TABLE No. 1.

SPAN IN FEET.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Span..... | 1 | 2 | 3 | 4 | 5 | 10 | 20 | 30 | 40 |
| Per cent..... | 44 | 63 | 77 | 90 | 100 | 71 | 60 | 41 | 36 |
| Span..... | 50 | 60 | 80 | 100 | 120 | 160 | 180 | 200 | 300 |
| Per cent..... | 32 | 29 | 25 | $22\frac{1}{2}$ | 20 | 17 | $16\frac{1}{2}$ | 16 | 13 |
| Span..... | 400 | 500 | 600 | 800 | 1200 | | | | |
| Per cent..... | $11\frac{1}{2}$ | 10 | 9 | 8 | $6\frac{1}{2}$ | | | | |

This rule gives 100 per cent. increase for spans of 5 feet. Below 5 feet the increase given by the rule will be more than 100 per cent. This is clearly wrong, for nothing over 100 per cent. is possibly correct. Hence, whilst for spans of 5 feet and upwards, the formula for percentage is $p = \frac{100 \times T}{t}$; below 5 feet, or for any other deflection and speed below such span as requires an addition of 100 per cent., the formula becomes $p = \frac{100 \times t}{T}$, $T$ being the time necessary for a body to fall through a distance to the deflection, and $t$ being the time occupied by the load in traversing a distance equal to half span. The inversion of the fraction $\frac{T}{t}$ is clearly rational, for in small spans the time occupied in attaining mid-span is less than that necessary by the law of gravity for the load to fall a distance equal to the deflection, and the percentage addition cannot be 100 per cent. Seeing, however, that all loads will not cross short spans at maximum speed, no span less than 5 feet ought to have a less than 100 per cent. addition. If this reasoning be correct, it would appear that for high speeds the stress in rails supported at more frequent intervals than 5 feet will be much less than under slower moving loads. Clearly for the speed and deflection employed in this paper 5 feet is the most severely tried span, as its half length time of load passage just coincides with the period of fall due to gravity. For higher speeds the limit of most severely tried span is lengthened, and it is quite possible to conceive that a bridge of 100 feet span could be so constructed, as regards the gradient of its rail surface, that a load moving at an immense velocity would cross from the higher to the lower abutment without exerting pressure on the bridge. Of course such necessary velocity is far beyond the range of even an improbable practice, for if one abutment was 1 foot lower than another and the span 100 feet, the velocity would require to be 400 feet per second, or 270 miles per hour. Thus at 300 miles per hour bridges less than 100 feet span might be omitted, so long as the far side abutment was a foot below the near side.*

To resume, however, such would complete the additions to be made to the load, and this second addition would prudentially be applied to the already augmented engine concentrations.

* At this high speed it is curious to reflect that any grade steeper than 52 feet to the mile could not safely join a horizontal piece of line without an intermediate flatter gradient. This statement, of course, neglects consideration of springs.

The method would also apply to the more detailed stresses in floor systems.

Stringers should be figured as though short-span bridges and the proper increment due to their deflection should be added. The suddenness of application of loading of floor beams is determined in each case by the stringer length and the percentage to be added is that proper to a span of a length equal to two panels of the bridge.

It will be correct also in figuring for stringers to increase the wheel loads, already determined by the above two additions, by the further addition due to lateral swaying of the whole machine, or by the amount of transferred load due to wind pressure. The maximum possible on either of these is 100 per cent., for beyond this an engine will overturn. What is the actual percentage due to a 30-pound wind pressure depends upon the type of engine, but the writer does not know what this may be; probably surging and wind combined may add in practice 25 per cent., which would be figured upon the already increased wheel loads. The final drive wheel concentration upon a 20-foot stringer will, with this last addition, be double its static amount. The maximum floor beam stress will be similarly found, but will be little or nothing affected by this last addition, which is a subtraction from the windward stringer of a pair. Truss stresses generally are to be calculated with this transferred load increment, which for the train loads may be 100 per cent., and in place of wind stresses being as now looked upon as extraordinary loading, future requirements with increased nominal working stresses will demand their equality with other stresses, the maximum being taken as the basis of design. The increments due to velocity of approach, whilst depending on panel length in some cases, such as beam hangers, etc., do not do so in all cases. For example, the rivet connection between a stringer end and a floor beam web plate is exposed to an instantaneous load at one end at whatever speed and must be allowed for accordingly. Of the main trusses the portions exposed to sudden loading are the counter-braces, and the additive percentage for these clearly depends upon their distance from the truss end.

Thus in a through bridge the distance from the approach end of the truss to the foot of any counter-brace will be the determining length and the percentage to be added will be that due to a span of double this length. (See foot note to Table No. 3, relative to Pratt trusses.)

If this principle be correct it will be clear that the addition

necessary for quickly applied load is greater than is often thought necessary and decreases with the decrease in the deflection allowed. Deep trusses are thus *per se* stronger than those of small depth, owing to their less deflection.

Of the secondary causes of load increment—those of structural origin—that due to the centrifugal action following on deflection below the horizontal is usually provided for by initial camber. That due to deflection of floor stringers should clearly be eliminated as far as possible by the use of deep and stiff stringers, and any deflection of the stringer then existing may be counteracted as regards panel point shocks by an upward camber of the track in each panel length, though this latter appears to be a refinement impossible to carry out in daily work. Its effect would be to diminish the jar and the tremor induced by the rise and fall of each wheel as it passed from the deflected mid-span of the stringer to the higher level over the floor beam.

The engines upon which American bridge loads are calculated appear to have drive wheel circumferences of about 13 feet, and therefore such engines will cause tertiary stress increments from the coincidence of revolution of wheel with floor beams when panels are 13 feet long, such increments being greatest when the lowest positions of counterweight coincides with the floor beam, and least when it drops upon the stringer center, as the centrifugal action then lifts the wheel over the panel point. This applies only to general truss stresses; the reverse is the case with the stresses in the stringers, except for their end shear and connections; 13 feet, 16 feet and 21 feet would appear to be a usual drive wheel circumference in American engines, and such lengths should be avoided as panel lengths, which, with such wheels, should not be of the following lengths: $6\frac{1}{2}$, 8, $10\frac{1}{2}$, 13, 16, 21, 26 or 32 feet.

With those three common sizes for wheels a panel length is hard to find which shall suit each wheel, and by preference one should be selected to suit that engine in which $F$ is greatest. Usually this will be the case in the 16-feet wheels which run at high speeds. The 13-feet wheels of consolidation engines do not revolve quickly, hence $F$ is small. Panel lengths of 9, 14 and 23 feet will be suitable, preferably the longer. Indeed it may be laid down as a rule that the panel length should exceed by two or three feet the circumference of the largest wheel, or failing this be intermediate between this length and the circumference of the next smaller wheel when such can be arranged. The panel lengths should

also not coincide with the wheel spacing of the train, for the united coincidences of car wheels and panel points will be more harmful than the coincidences of the counterweight action. When such coincidences must be allowed to occur a heavy percentage will need adding to train load, and further experiments ought to be carried out on the lines of those of Professor Robinson to determine what such percentage will be.

The cumulative vibration due to fortuitous agreement of the above coincidence with the periodic oscillation of the bridge itself will be sufficient to warrant every care being taken to prevent the first coincidence, so that the door may not be left open for the even possible triple or rather quadruple coincidence.

With careful design, bearing in mind the above, it would appear from the experiments already tried that about 25 per cent. at least should be added to truss stresses on account of cumulative or partially cumulative vibratory action.

Further experiment should be in the direction of determining the oscillation period of standard types of bridge of varying span, in order that the train speed may be known and avoided which will give maximum cumulative effect. The subject is one of difficulty, depending as it does on so many factors. The introduction of a single vehicle in a train having a wheel base different from other vehicles might be the means of saving or destroying a bridge; for, however improbable may be the chance of unity of action of all causes of load increment and vibration, such might occur to any bridge at some time and when occurring might wreck it.

To sum up, it would seem that primary causes of load increment affect chiefly the floor system, as also do the secondary effects due to structural peculiarity, though these also affect trusses. The greatest truss effects, except as regards counters which are primarily affected, are due to the tertiary or combined effects of load and structure.

The method of computing impact percentages on the time basis appears to be correct. It is on the safe side as compared with the inverse method. For example again, taking the case of the 180 feet span we have a deflection $= \frac{9}{20}$. The time of traversing the half span is one second. In one secomd 16 feet would be fallen through. Then $\frac{100 \times \frac{9}{20}}{16} = 2.8$ as the added percentage for the speed and span. Calculated on this basis the percentages for a few spans will be as follows:

TABLE No. 2.

| | | | | | | |
|---|---|---|---|---|---|---|
| Span, feet | 5 | 10 | 20 | 30 | 40 | 50 |
| Per cent | 100 | 50½ | 25 | 17 | 12½ | 10 |
| Span, feet | 100 | 150 | 180 | 210 | 300 | 400 |
| Per cent | 6 | 3½ | 3 | 2½ | 1¾ | 1½ |

This method agrees with the first method at the 5-feet span above, and gives much less percentage additions for long spans, especially for deflections to less than the $\frac{1}{100}$ here supposed.

It does not differ much from the table given in "Waddell's Japanese Memoir" for spans up to 60 feet only, but the first method would appear to be preferable, showing as it does large percentage additions to spans under 25 feet, and reasonable additions for long spans. Waddell's memoir, too, deals with practical deflections and agrees with a modified Table No. 1.

It will be noted that structural weight does not enter into the calculations, and further investigation would seem necessary to determine whether the relative proportions of dead and rolling load have an effect in reducing the necessary impact addition. The writer does not think any allowance can be fairly made for weight of structure. Should an allowance be made it would, of course, act chiefly in diminishing the impact addition for the longer spans, in which the ratio $\frac{\text{dead}}{\text{live}}$ is a larger number than in the case of the short spans.

The maximum deflection due to live load being the measure of the work done, however, it would seem that no allowance should be made, and it would be of interest to measure the deflections of a bridge at every panel point with loads at various speeds and with engine drivers in and out of vertical balance, as well as with drive wheels of a circumference exactly equal to panel lengths.

The train, too, might be loose coupled, so that all wheels were simultaneously at a panel point. The deflection increments, at speed, above those with steady load, ought to give the increment of load which should be provided in stress calculations, but for maximum increment it would, of course, be necessary to find the bridge vibration period and harmonize the train speed to produce cumulation. The total increment for high speeds should then include not merely the maximum cumulative stresses, but the maximum stresses due to impact, surging and centrifugal action of drive wheel counterbalances. On the assumption that the experi-

mental investigation of cumulative vibration shows a 25 per cent. addition, and that for floor systems the transferred load due to surging and wind pressure is 100 per cent., the following table, No. 3, would exemplify the various actual calculable loads, after a first addition to drive wheel concentration of the amount due to centrifugal action, which should be taken in a bridge made up of nine panels of 20 feet, the amounts of percentage being taken from Table No. 1. The transferred stresses for trusses are figured as for an increment $(1 + x)$ written $1.x$, as the increment can only be found when the relative distance apart of rails and trusses is known, and for the stringers the increment is taken at 100 per cent. or the overturning point of the train. The transferred locomotive load would, as already stated, be much under this amount. Post stresses require an increment equal to that of the counter-brace attached to the upper end, in the case of through bridges, and the stress thus determined must be compared with that due to truss general stress and the greater of the two employed in determining sections. Probably the most expeditious way of finding stresses by this method will be to find structural load stresses separately and determine rolling load stresses as though steady, afterwards multiplying them by the factor under the head of Total in Table No. 3. These stresses being determined, the working stress per square inch of the material is to be found in the usual way from the ratio of minimum to maximum load.

TABLE No. 3.

| | Static. | Impact. | Cumulation. | Transferred. | Total equivalent static. |
|---|---|---|---|---|---|
| Trusses, general stresses.... | 1.00 | 1.165 | $1.165 \times 1.25 = 1.45$ | $1.45 \times 1.x =$ | $1.45 + 1.45\,x$ |
| Hip verticals and beam hangers—1st Counter-brace ..... | 1.00 | 1.36 | $1.36 \times 1.25 = 1.70$ | $1.70 \times 1.x =$ | $1.70 + 1.7\,x$ |
| 2d Counter-brace* ..... | 1.00 | 1.25 | $1.25 \times 1.25 = 1.56$ | $1.56 \times 1.x =$ | $1.56 + 1.56\,x$ |
| 3d Counter-brace* ..... | 1.00 | 1.20 | $1.20 \times 1.25 = 1.50$ | $1.50 \times 1.x =$ | $1.50 + 1.5\,x$ |
| 4th Counter-brace* ..... | 1.00 | 1.17 | $1.17 \times 1.25 = 1.46$ | $1.46 \times 1.x =$ | $1.46 + 1.46\,x$ |
| Stringers, general..... | 1.00 | 1 60 | $1.60 \times 1.25 = 2.00$ | $2.00 \times 2.0 =$ | 4.00 |
| Stringers, end shear, etc. | 1.00 | 2.00 | $2.00 \times 1.25 = 2.50$ | $2.50 \times 2.0 =$ | 5.00 |
| Floor beams..... | 1.00 | 1.36 | $1.36 \times 1.25 = 1.70$ | $1.70 \times 1.x =$ | $1.70 + 1.7\,x$ |

The method of this essay clearly gives larger maximum stresses than custom assumes; more especially is this noticeable in stringer end connections. If the method be correct, it is very clear that short spans, as

* For counter-braces past the middle of span the percentage is clearly that due to half span distance from approach, as they then take general truss stress. Though only commencing from second panel point in Pratt trusses, the span so far as regards counter-braces also only commences at the hip vertical.

now built, are relatively weaker than long spans, and good modern practice tends to support the accuracy of the factors herein determined. For floor systems, etc., the method is clearly more accurate than for trusses, as the loads of floor stringers more nearly approximate to the condition of a single load than in the case of trusses, and in very short panels are identical. For the larger spans, too, the assumption that maximum deflection takes place when the head of train reaches half span is an error on the safe side, and the percentages for impact might probably, with a near approach to accuracy, be halved for spans of 400 feet and correspondingly reduced down to the 25-feet span, for which no reduction of impact percentage need be allowed.

The writer would not make any claim to finality in this method, but offers these few remarks in the hope that they may be of service as indicating at least a line of approach towards the true solution of the real stresses due to moving loads, which, if once determined with accuracy, would engender a confidence and precision in design hitherto quite impossible.

Experimental proof is clearly required to determine actual increments, and the measure of deflection appears to present the easiest method of doing this.

The deflection assumed, $\frac{1}{1000}$ of span for live load alone, is large. For deflections of less amount the percentage additions of Table No. 1 will be reduced in the proportion $\sqrt{\frac{d_1}{d}}$; $d_1$ being the amount of the new deflection.

Half the deflection would give $\frac{1}{\sqrt{2}}$ times the percentage. Expressed in a simple form, $P = \frac{50\ V\sqrt{D}}{s}$, when $P$ = percentage addition; $V$ = velocity of train in feet per second; $D$ = deflection due to live load in feet, and $s$ = span in feet. The large additions made to live loads by the percentage factors serve to raise the value of the maximum $B$ in the formulas of Wohler, whilst leaving minimum $B$ unaltered in most cases.

Hence, the working stress allowable is lowered, but when all such percentages have been added and the ultimate strength of every detail determined from the known ratio $\frac{\text{Maximum } B}{\text{Minimum } B}$, no factor remaining unknown, it may fairly be claimed that a factor of safety from 2 to 3 will be all that is necessary; sufficient, in fact, to bring the work of every part within its elastic limit.

The experiments of the Iron Commissioners in 1849 showed that in certain cases deflections under moving loads were over double of those due to the static load. This does not prove that in any case more than 100 per cent. addition is required, but rather that in the special experiments the true elastic limit had been exceeded, for the amount of deflection in the short bar experiments was $1\frac{1}{2}$ inches in 9 feet, or $\frac{1}{71}$ of the span.

The immediate conclusions to be drawn from the arguments of this paper are that a double-track bridge is better than two side by side bridges of single track, owing to maximum stresses being much less frequent, and to the decreased possibility of maximum stresses due to transferred load under wind pressures ever taking place at all; but it is evident that in approaching to the true working limit of a material under absolutely known loading, the maximum possible stress only can be admitted into calculations, and the present specified allowances of 25 and 50 per cent. greater stresses under extraordinary loadings, such as wind, will be disallowed in the near future, or considerably modified.

Since writing the foregoing I have had sent me a copy of Mr. S. W. Robinson's paper on bridge vibrations. From this paper I observe that an ordinary deflection of bridges is about $\frac{1}{240}$ of span. My table No. 1, giving impact percentages for an assumed deflection of $\frac{1}{400}$, may be fitted to any other deflection very readily, impact varying, as $\sqrt{D}$. Hence for deflections of $\frac{1}{2400}$ the tabular numbers must be multiplied by $\frac{1}{\sqrt{6}}$ or by $\frac{1}{2.45}$, which would give just about 10 per cent. for spans of 80 feet and about 5 per cent. for 300 feet, whilst for the 20-feet span about 25 per cent. appears the proper addition. The deflection of a Bollman truss appears to be about $\frac{1}{800}$, or half the assumed $\frac{1}{400}$. Hence, for Bollman * and similar trusses the allowance for impact will be $\frac{1}{\sqrt{2}}$ of the tabular numbers, or nearly $\frac{3}{4}$. From this it is clear that whilst such trusses are not exposed to very cumulative vibrations, yet they are exposed to an impact nearly double that of Pratt trusses. As cumulative vibration appears worst in the case of long spans, the first conclusion to be drawn would appear to be that the Bollman truss is eminently suited for long spans. Deeper thought, however, leads to a different conclusion. With a $\frac{1}{2400}$ deflection, a 300-feet Pratt truss re-

* The Fink truss is an improvement upon the Bollman in respect to impact.

quires 5 per cent. for impact, and, according to Professor Robinson, 50 per cent. for cumulation, or a total of 57½ per cent. Now, the Bollman truss of 300 feet may have, perhaps, 5 per cent. only of cumulation, but as regards impact not the 9 per cent. which .7 of the tabular number 13 would show, and why? Because the Bollman truss is really composed of several independent trusses, having one chord only in common, and a bridge on this principle having a span of 300 feet and 25 panels really consists, so far as impact is concerned, of a series of 40-foot bridges, each having a deflection of $\frac{1}{300}$ of the whole span. Thus, with a deflection equal to that of a 300-feet span, the impact is that of a span whose half length is equal to a panel length of the truss. Table No. 1 gives 36 per cent. for spans of 40 feet, and three-fourths of this is 27 per cent. Hence the total addition to the Bollman truss is 133 per cent., on the supposition that cumulation adds so little as 5 per cent. The Bollman truss is therefore seen to possess no solid advantage, even on this score, over Pratt trusses, and it seems probable that attempts to stiffen such trusses by interpanel bracing may serve to increase cumulation stresses, whilst doing little or nothing to diminish the effects of impact. Long panels would appear an essential in all such types of bridges, in order to diminish impact.

Allowing 2 pounds per square inch of piston area for weight of piston, rods, etc., an ordinary 18-inch engine would figure out to about 500 pounds. Taking half this as reciprocating, the centrifugal action of the vertically unbalanced driver is $F = .00034\ W\ r\ N^2$ ; $r$ being taken equal to crank-pin radius, or 1 foot ; $N$, for a 66-inch wheel, at 90 feet per second, is 312, and $F = 8\,274$ pounds, or a difference of 16 548 pounds, which difference is brought to bear on the truss or rail in half a revolution of the wheel, or in $\frac{1}{10.4}$ of a second. Now, one revolution of a wheel is 17.27 feet, and $\frac{1}{2400}$ of this is $\frac{1}{139}$ of a foot deflection, assuming local deflection to occur. The time occupied in falling freely through $\frac{1}{139}$ of a foot is $\frac{1}{47}$ of a second, hence $100 \times \frac{1}{47} \div \frac{1}{10.4} = 22$ per cent. to be added to driver counterbalance excess for static equivalent, practically a fourth. In the present instance the driver excess becomes fully 10 000 pounds above mean load, or 20 000 pounds difference. This is of itself equivalent to an addition of 18 per cent. to the mean weight of a standard 112 000-pound passenger engine, whilst the centrifugal difference is 36 per cent., which, after allowing for speed differences, is curi-

ously near the amount of engine percentages found by Professor Robinson, and though the means of finding this may be scarcely scientific, I am inclined to the belief that it is to this, rather than to cumulation, that engine excess is to be thought due, though in the case of train excesses the panel coincidence holds good. I should be inclined to believe also that when counterbalance excess is down at floor beams and up at mid-panel the effect would be in certain cases as serious as when the reverse is the case. Though for general stringer stresses counterweights down at mid-span may be productive of the worst effects of impact, yet for stringer connections to floor beams the centrifugal action, when the weight is down at stringer ends, *i. e.*, at floor beams, added to an already found 100 per cent. for instantaneous application as the wheel comes from over the floor beam, must be very serious. As, however, it is impossible to choose how a locomotive shall place its wheels, allowance must be made in every case for most disadvantageous position.

The percentage due to centrifugal action requires to be added to the engine loads to an extent at 60 miles of probably 40 per cent., as deducible from Professor Robinson's figures for lower speeds, before the addition of the further excess for impact. The fact that centrifugal action varies with the square of the wheel revolutions is presumptive evidence that engine vibration for freight trains should, as observed, be small. It is also observable that, whilst in passenger trains the locomotive is the cause of vibrations, in freight trains the cars form the chief stress-producing element, and this is to be taken as showing that a bridge is likely to be safe under a passenger train if calculated for a freight train vibration, and further, that large wheels to an engine are conducive to safety.

Locomotives might be perfectly counterbalanced if somewhat differently arranged as regards cylinders. To balance the pistons an arrangement of four cylinders with the two pistons on the same side of engine, moving in opposite directions, might be arranged on an overhung double crank, or the counterweight excess of present engines might be balanced by the introduction of a vertically moving balance driven by connecting rod from the crank or wrist pin. The subject is worth more attention than has been yet accorded it, for it is certain that the vibrations set up in the ground and contiguous buildings to a railway by the unbalanced counterweights at high speeds must act prejudicially on bridge structures.

PLATE XVIII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

412.
(Vol. XX.—May, 1889.)

# THE CONSTRUCTION OF THE BRIDGE OVER THE ARKANSAS RIVER, AT VAN BUREN, ARKANSAS.

By C. D. PURDON, M. Am. Soc. C. E.

## WITH DISCUSSION.

The following paper is the substance of a report made by the writer to Mr. James Dun, M. Am. Soc. C. E., Chief Engineer of the St. Louis and San Francisco Railway, upon the construction of the bridge over the Arkansas River, at Van Buren, Ark.

A great deal of difficulty and delay was experienced at first from the unusually high water in the river, in which it may be said there was no intermission during the Spring of 1885, until July, and in fact until that time but little work on the actual construction was accomplished.

We found that 10 feet above low water seriously interfered with any work on foundations and 15 feet proved enough to stop work altogether.

By reference to the profile (Plate XXIV) showing the stage of water during the year, it will be noticed (the first work, viz., building caissons for Pier 2, having been commenced April 10th), that in April, May, June and July we had at different times enough high water to stop all work, and the intervals between the rises were so short that but little could be accomplished. A rise also took place in September which stopped all work except on Pier No. 1; this rise, however, did not last long.

But little difficulty was experienced in sinking any of the caissons, as will be observed by reference to the diagram showing the sinking (Plate XXXV), and the rock was found in all piers, except No. 2, to be almost level. A good deal of trouble was encountered with the caisson of Pier 3; also the sinking of Pier 1 was delayed by bowlders.

The character of the bed rock was good; it consists of a hard, clayey slate, which is in places overlaid by an inch or two of very tough blue clay, with which the seams of the rock are also filled; it is also in some places overlaid by bowlders which are very hard, apparently of clay and sand mixed, of a bright blue color, and showing specks of mica or iron pyrites.

The average of the elevations of the rock at the different piers as finally established was almost identical with the original borings, piers 2 and 5 showing the greatest difference. This is shown by the appended statements for each pier.

The timber used was yellow pine, from William Farrel's mill at Wrightsville, Ark., and was of excellent quality. All the timber used, except bridge ties, came from this mill, and cost $9, $10 and $11 per thousand feet, board measure, at the mill for 20, 30 and 40 feet lengths. The freight (172 miles) was about $4 per thousand.

Of the stone used, 366 cubic yards came from Mountainburg quarry, about 20 miles north of Van Buren. This stone was used in Piers 1 and 10 only, as it was judged too soft to put in piers exposed to a strong current. It is a somewhat yellow sandstone, showing traces of iron, and weighs 139 pounds per cubic foot.

Two hundred and thirty-eight cubic yards of limestone were quarried near Garfield, Ark. It is of a blue color, very hard, and weighs 158 pounds per cubic foot. It was used for the lower courses of Piers 2 and 4.

All the rest of the stone came from three quarries near Beaver station, on the Eureka Springs Railway. This is a white limestone of excellent quality, and weighs 155 pounds per cubic foot.

A small quantity of backing was quarried about three-quarters of a mile above the bridge site on the north bank of the river, where also was quarried all the stone crushed for concrete and that used for riprap on Piers 7, 8 and 9.

Pier 1 was at first designed to be built in an open caisson; but for reasons stated hereafter under the head of Pier 1, a caisson sunk by the pneumatic process was substituted.

Piers 2, 3, 4, 5 and 6 are each built on a caisson sunk by the pneumatic process.

On all these caissons a crib was built, filled with concrete. This crib was designed to be of such a height as to be 4 feet below low water when the caisson finally rested on the bed rock.

The working chamber of all these caissons was filled with concrete after sinking. This concrete was carefully rammed in place under the personal inspection of members of the engineer corps, who relieved each other in this duty.

After the caisson had been filled in this manner, a quantity of grout was poured in, under air pressure, and the air pressure was continued from twelve to eighteen hours after grouting.

The greatest pressure used during the sinking was 24 pounds per square inch, which was while filling the caisson at Pier 2, the water then being high.

Piers 7, 8 and 9 are each built on a grillage supported by piles. This grillage is described in its appropriate place.

These piles were driven by a hammer weighing 4 700 pounds, assisted by a jet of water, and some of them required over three hundred and fifty blows, the average being about two hundred and eighty. They were all driven to refusal, and are believed to be on rock, as the length of the piles agrees with the level of rock given by borings. They were of various kinds of oak, hickory, elm and gum, and a few sawed sticks of 12 x 12 pine were used.

Pier 10 was built on a concrete foundation in the usual manner.

The masonry generally was also delayed by high water. Pier 10, which was rather advanced than retarded by high water, was built in May and June. Pier 2 was built in June and July, and all the rest of the masonry between the 11th of August and the 21st of November.

In the latter piers two gangs were employed. The average number of cubic yards laid by each per working day was 21.95.

The masonry is of excellent quality, both as regards stone and workmanship, and although some bridges may excel it in exterior finish, we think it will compare favorably for solidity and workmanship with any railway masonry in the country.

But two deaths occurred during the building of the substructure; one laborer employed in the pneumatic work was drowned by falling off

a barge; and a sub-foreman in the employ of the Union Bridge Company died of congestive fever.

A carpenter had his leg broken by a flying piece of a stump, which was dislodged by a blast in the quarry near the bridge site.

The caissons were built and sunk by William Sooysmith & Son, of Chicago, under a sub-contract from the Union Bridge Company, and they showed great energy and skill in their work.

The masonry was built by the Union Bridge Company, under the superintendence of C. E. Danforth.

The superstructure consists of two spans 252 feet 9 inches, center to center of end pins, north of the draw.

One draw span, rim bearing, 366 feet, center to center of end pins.

One span 252 feet 9 inches, and four spans 162 feet, center to center of end pins, south of draw; the total length of bridge over all being 1 798 feet.

Considerable delay occurred on account of the iron not coming forward at the proper time. For instance, the pivot pier was finished October 5th, but no iron was received for the span until December 13th.

Pier 2 was finished July 20th, and Pier 1 October 13th. Enough iron for this span (No. 1) was not received until November 4th.

Pier 10 was finished June 9th, and Pier 9 August 29th. Enough iron for this span was not received until December 12th.

Pier 8 was finished September 30th, and Pier 7 November 9th. The iron for these spans was not received until December 19th.

The spans are all single intersection, 18 feet wide in the clear. The 252 feet 9 inches spans and the draw have inclined top chords. The 162 feet spans have parallel chords.

The last iron for the draw span was received January 22d, 1886, and the erection begun the 25th. The trusses were all coupled February 1st, and the first train, No. 17, south bound, passed over the bridge at 3.30 P.M., February 9th, 1886.

### Pier 1

Is the north abutment of the bridge. According to the contract, this pier was to be built on a bed of concrete 18 by 29 feet, built up from the bed rock (elevation given, 350 feet, plus datum) to low water (362.7) inside an open caisson, or an open space inclosed with sheet piling.

On April 14th, 1885, the pit was staked out and excavation begun, to be stopped by high water on the 22d. It then remained untouched until June 16th, when it was again started, to be stopped by high water on the 18th.

After this, on account of the high bank behind the pit, and the difficulty already experienced from high water, the Union Bridge Company agreed with Sooysmith & Son (the contractors for foundations) to build and sink a caisson and crib for the pier, size 20 by 30 feet.

Accordingly, on August 14th, the caisson was commenced, being built in position, and before being roofed over was sunk as far as possible by shoveling. The caisson and crib (4 feet of crib) being finished, the crib was filled with concrete, and an air pipe laid from the pressure boat, which was then at Pier 3. A rise in the river prevented this plan of sinking being carried out, and as the river became too high to continue work on Pier 3, the pressure boat was moved over to Pier 1, and sinking commenced September 10th.

After the cutting edge had reached the depth of 9 feet below low water a mass of bowlders was encountered which had to be broken up by blasting and other means and a large amount of them brought out through the air lock, which greatly delayed the sinking to bed rock; this was finally reached September 25th.

On the concrete in the crib the masonry was commenced, and consists of three footing courses each 2 feet high, with 9-inch offsets. On these the neat work begins, 13 feet 10 inches by 24 feet, and is carried up with plumb sides, face with a batter of 2 inches to 1 foot; and back plumb with three 9-inch offsets. The coping is 7 by 25 feet, and at the back of it is the ballast wall 24 feet by 2 feet 6 inches and 5 feet 4 inches high.

The first footing course is of Eureka Springs limestone. Above this the pier is of Mountainburg sandstone up to 7 feet below the coping. The rest of the pier is limestone.

The elevation of bed rock proved to be 351.15.

| | |
|---|---|
| Began excavating pit (1885) | April 14th. |
| Stopped by high water | " 22d. |
| Began again | June 16th. |
| Stopped by high water | " 18th. |
| Began building caisson in place | August 14th. |
| Finished building caisson and crib | September 3d. |
| Began concrete in crib | " 3d. |
| Finished concrete in crib | " 4th. |
| Began sinking | " 10th. |
| " masonry | " 11th. |
| Finished sinking and began filling chamber | " 25th. |
| " filling chamber | " 27th. |
| " masonry | October 13th. |

QUANTITIES AND WEIGHTS.

| | | |
|---|---|---|
| Total contents of masonry | 391.7 cubic yards. | |
| 88.3 cubic yards limestone, at 155 pounds per cubic foot | 369 535 | pounds. |
| 229.2 cubic yards sandstone, at 139 pounds per cubic foot | 860 187 | " |
| 36 712 feet B. M. timber, 4½ pounds per foot | 165 204 | " |
| Concrete in chamber 68.3 cubic yards. | | |
| " crib 66.3 " " | | |
| " air shafts 11.9 " " | | |
| Total 146.5 at 130 pounds per cubic foot, | 514 215 | " |
| Iron | 4 128 | " |
| 222 barrels cement, at 290 pounds | 64 380 | " |
| 315 barrels sand, at 375 pounds | 118 125 | " |
| Total | 2 095 774 | " |
| Less displacement, 20 x 30 x 11.55 cubic feet | 431 947 | " |
| (Water taken at 62.33 pounds per cubic foot.) Net | 1 663 827 | " |
| Area of foundation | 600 square feet. | |
| Pressure from substructure: | | |
| Per square foot (pounds) | 2 773 | |
| " square inch " | 19.26 | |
| Superstructure and Load: | | |
| 252 feet 9 inches truss, dead load, 2 536 pounds per foot. | 640 974 | |
| " " " live load, 3 000 pounds per foot, | | |
| + 40 000 excess | 798 250 | |
| | 1 439 224 | |

Half of which belongs to Pier 1 = 719 612.

| | |
|---|---|
| Pressure on Foundation from Superstructure and Load: | |
| Per square foot (pounds) | 1 199 |
| " square inch " | 8.32 |
| Total weight on Foundation: | |
| Per square foot (pounds) | 3 972 |
| " square inch " | 27.58 |

PIER 2

Is built on a caisson sunk to bed-rock. The dimensions of the caisson are 18 x 39 feet over all. The working chamber is 6 feet high, with walls of two thicknesses of 12 x 12 pine, sheeted inside and outside, with 3-inch plank. The roof consists of two courses of 12 x 12 pine.

On this caisson is built a crib of the same outside dimensions. The walls are of one thickness, of 12 x 12 pine, sheeted outside with 3-inch plank. It has one longitudinal brace in each alternate course, and four cross-braces in each course between the longitudinal braces, all 12 x 12, and dovetailed into walls. This crib is filled with concrete.

In the preliminary borings, the elevation of the bed rock at this pier was given as 342.5, and the caisson and crib were accordingly built 16 feet 6½ inches high, so as to commence the masonry 4 feet below low-water. During the progress of the work the elevation of the rock was found to be much lower, so the masonry was commenced larger than the original plans, to allow for the increased depth. When the caisson reached the rock, the rock was found to have a fall down stream of 7 inches in the length of the chamber. The up-stream end rested on the rock, also the sides in various places. The rest of the shoe was wedged up on 12 x 12 blocks, and concrete rammed under the shoe in the spaces between these blocks.

The elevation of the cutting edge, when finally resting on the rock, was 338.47, and of the top of the concrete in the crib 355.0. At this elevation the masonry was commenced. It consists of two footing courses 18 inches high and one 12 inches high, the sizes being respectively 12.11 x 35.2; 12.1 x 34.2, and 11.1 x 33.2, all being built with square ends. On this the neat work begins 10 feet wide and 22 feet long between shoulders, with semicircular ends.

The original plan was to carry up this masonry with a batter of ½ inch per foot, finishing 6 feet 6 inches wide under coping; but plans of the pedestals of the fixed spans having been received from the Union Bridge Company, which showed that this size would only allow 1½ inches for inaccuracy in location, sinking, etc., it was decided to carry up this batter to 2 feet below the coping, finishing 6 feet 8 inches wide at that point, and then reverse the batter and make it 3 inches per foot; then finishing under coping 7 feet 8 inches wide.

The coping consists of a 16-inch course projecting 6 inches.

A small part of the backing was quarried at the bridge site; all the rest of the stone came from Eureka Springs quarries, except a small part from Garfield quarry.

There were some delays from high-water, and from want of suitable stone; the building of the caisson was also delayed by high-water.

| | |
|---|---|
| Began building caisson (1885) | April 10th. |
| Launched caisson | May 13th, 1.45 P.M. |
| Located caisson | May 13th, 5.20 P.M. |
| Finally located | June 4th. |
| Began concrete in crib | " 3d. |
| Finished concrete in crib | " 5th. |
| Began sinking | " 6th. |
| Began masonry | " 7th. |
| Finished sinking | " 22d. |
| Began filling working chamber | " 23d. |
| Stopped work; water over shaft | June 27th, 8 A.M. |
| Began again filling | July 1st, 3 P.M. |
| Finished filling | July 2d, 4.20 A.M. |
| Stopped masonry for want of stone | " 4th, 9.30 A.M. |
| Began again | " 15th, A.M. |
| Last coping stone set | " 18th, 6.50 P.M. |
| Coping grouted and pier finished | " 20th, 7.45 A.M. |

QUANTITIES AND WEIGHTS.

| | | | |
|---|---|---|---|
| Solid contents of masonry | | | 450.4 cubic yards. |
| 387 cubic yards limestone, 155 pounds per cubic foot | | | (pounds) 1 619 595 |
| Concrete in chamber | 103 | cubic yards. | |
| " crib | 144 | " " | |
| " shafts | 14.2 | " " | |
| | 261.2 | " " at 130 pounds per cubic foot, | 916 812 |
| 55 568 feet B. M. timber at 4½ pounds | | | 250 056 |
| Wrought and cast-iron in foundations | | | 5 727 |
| Wrought-iron in masonry | | | 639 |
| 230½ barrels cement, at 290 pounds | | | 66 845 |
| 321 " sand, at 375 pounds | | | 120 375 |
| Total | | | 2 980 049 |
| Less displacement at low water | | | 894 402 |
| Net | | | (pounds) 2 085 647 |
| Area of foundation | | | 702 square feet. |
| Pressure on foundations: | | | |
| Substructure, per square foot | | | 2 971 pounds. |
| " per square inch | | | 20.63 " |

Superstructure:

252 feet 9 inches truss, dead load, 2 536 pounds per foot and
" " " live load, 3 000 " " + 40 000
excess = 2 050 per square foot, or 14.22 per square inch.
Total, 5 021 per square foot; 34.85 per square inch.

PIER 3

Is also built on a caisson similar to Pier 2. This pier was very unfortunate in its career. When launched (June 11th) the river was rising a little, and it was held at the launching place to put on part of the crib. It was started for its position on the 18th, towed by the transfer boat "Harold B." On passing the locating piles they failed to get a line out, and the boat being unable to manage the caisson, it grounded on the sand bar about half a mile below the bridge site. In the evening a rope was run from it to the locating piles, and the hoisting engine put to it. The only effect of this was to break off two piles. After this all the available steamboats, viz.: "Harold B," "Poteau," "Tempest," and the ferry-boat tried to tow it up, but only succeeded in getting it afloat. Late in the evening they took hold of it again, and finally succeeded in getting it into position at 2 A. M. on the 19th. On the 20th it was located and concrete in crib begun, which was continued up to the 23d, when the river began rising very fast, and the sand scouring out from the up-stream end of the caisson; to protect it a row of piles was driven across the stream about 50 feet above the caisson, and brush cut and weighted down with stone above them. The caisson was now a good deal out of line, the up-stream end being low.

July 3d the river had fallen a little, and the pressure barge was brought over and air put on and a part of the false bottom got out, but on the 4th the river again began rising, and on the 5th the air had to be taken off and the boat moved away, as she was in danger from the heavy drift running. On the 7th so much drift had caught on the piles driven to protect the caisson that they broke off, and the mass of drift coming down against the air shaft (the caisson and crib being under water) broke it off at the top of the crib, and the upper section containing the lock was lost in the river. The caisson was now 8½ feet out of level in its length, and 2 feet in the direction of the bridge, the northeast corner being the highest. Nothing more was done until August 27th, as a caisson for another pier could be sunk quicker to give employment to the masons.

A good deal of doubt was felt as to whether this caisson could be used at all, as it had moved down stream, and should it pivot on its up-stream end in leveling, it would be so much out of place that the masonry could not be put on it. A plan was suggested to widen the up-stream end by driving a row of timbers 12 x 15 close to it and bolting them to the caisson by means of a diver, but it was finally thought most advisable to try and level the caisson by pivoting on the center and forcing the up-stream end to rise as it came level. A diver was employed to take away the drift and logs, and when they were removed the work was commenced.

The up-stream half of the false bottom was still in the caisson, and materially assisted this plan. The crib was finished, and the down-stream end filled with concrete and the work of leveling commenced. It was entirely successful, and on September 5th the caisson was practically level.

It was now found to be 2 feet 8 inches down stream, and 2 feet 1 inch south from its true position. The down-stream end and south side were accordingly kept as low as possible to work it back, and it was finally got back far enough to begin the masonry by leaving out the offsets on the up-stream end on the footings and beginning the first footing course 4 inches from the north side of the crib.

The masonry was begun September 8th, and two footing courses put on, when the river began rising again, and as it was obvious that the masonry could not be kept above the water and that the caisson might move, it was thought best to stop work and not commence the neat masonry until the river fell again. Accordingly work was stopped until September 27th, when pressure was again put on and the caisson brought level. The neat work was then commenced (September 29th), and all went well until the caisson touched rock October 5th. Filling the chamber then began, and was completed October 7th. Enough masonry was built to keep the pier above an ordinary rise, and it was then left until the 15th, when it was resumed and continued with a delay of a few days until its completion.

| | |
|---|---|
| Began building caisson (1885) | May 18th. |
| Launched " | June 11th, 4 P.M. |
| Located " | " 19th, 1 P.M. |
| Began concrete in crib | " " " |
| Stopped work | " 22d. |
| Drove piles up-stream for protection | " 24th. |
| Began pressure | July 3d. |
| Stopped for high-water | " 5th. |
| Drift carried away air-lock | " 7th. |
| Again began pressure | August 27th. |
| Began concrete in crib | September 6th. |
| Finished " " | " 7th. |
| Began masonry | " 8th. |
| Stopped for high water | " 10th. |
| Again began pressure | " 27th. |
| " masonry | " 29th. |
| Began filling caisson | October 5th. |
| Finished " " | " 7th. |
| Finished masonry | November 17th, 1.30 P.M. |

## Quantities and Weights.

| | | |
|---|---|---|
| Solid contents of masonry | 493.7 cubic yards. | |
| 433.9 cubic yards limestone, 155 pounds per cubic foot | (pounds) | 1 815 871 |
| Concrete in chamber | 91 cubic yards. | |
| " crib | 334.4 " " | |
| " shafts | 2.7 " " | |
| | 428.1, at 130 per cubic foot. | 1 502 631 |
| 77 029 feet B. M. timber, 4½ pounds per foot | | 346 630 |
| Wrought and cast-iron in foundations | | 7 378 |
| Wrought iron in masonry | | 1 192 |
| 264½ barrels cement, at 290 | | 76 705 |
| 377½ " sand, at 375 | | 141 562 |
| Total | | 3 891 969 |
| Less displacement | | 1 245 415 |
| Net | (pounds) | 2 646 554 |

Area of foundation ..... 702 square feet.

Pressure on foundations:

| | |
|---|---|
| From substructure, per square foot | 3 770.0 |
| " per square inch | 26.19 |

Superstructure and load:

| | |
|---|---|
| ½ of 252 feet 9 inches truss, dead load, 2 536 pounds per foot. | |
| " " " " live " 3 000 " " | |
| + 40 000 excess = | 719 612 |
| ⅔ of load on one arm of draw-span, 3 000 pounds per foot, | |
| + 40 000 excess = | 220 875 |
| Total | 940 487 |

From superstructure:

1 339 pounds per square foot; 9.30 per square inch.

Total, 5 109 per square foot; 35.49 per square inch.

### Pier 4.

This pier carries the draw-span, and is circular in shape, 30 feet diameter, and rests on a caisson sunk to bed rock.

The caisson is octagonal in shape, and measures 38 feet 6 inches across. The chamber has walls of two thicknesses of 12 x 12 pine sheeted inside and outside with 3-inch plank. The roof consists of four courses, 12 x 12 pine. On this is built a crib of the same outside dimensions as the caisson. The walls are single, 12 x 12 pine, sheeted outside with 3-inch plank. There are three ties 12 x 12 in each course of crib, alternating. This crib is filled with concrete.

The bottom footing is octagonal in shape and consists of two 12-inch courses of masonry, 33 feet across. The second footing is 24 inches high, circular in shape, and 31 feet 6 inches diameter. On this the neat work is built, 30 feet diameter. The coping is 31 feet diameter, 18 inches thick at center of pier and 16 inches at edges.

No difficulty was met with in sinking the caisson, and the bed rock was found to be not more than 2 inches out of level in the width of the chamber.

| | |
|---|---|
| Began building caisson (1885) | June 12th. |
| Launched caisson | " 23d. |
| Located caisson | " 30th. |
| Began concrete in crib | " 30th. |
| Began sinking | August 2d. |
| Finished concrete in crib | " 10th. |
| Began masonry | " 12th. |
| (No masonry built August 20th to 28th, while filling chamber.) | |
| Finished sinking | August 21st. |
| Began concrete in chamber | " 21st. |
| Finished concrete in chamber | " 27th. |
| " " shafts | August 27th, 11 P.M. |
| Resumed masonry | August 28th. |
| Finished " | October 5th. |

### Quantities and Weights.

| | | | |
|---|---|---|---|
| Total contents of pier | 1 001.8 cubic yards. | | |
| 827.8 cubic yards stone, 155 pounds per foot | | | 3 464 334 |
| 139 827 feet B. M. timber, 4½ pounds per foot | | | 629 221 |
| Concrete in chamber | 198.4 cubic yards | | |
| " crib | 710. " " | | |
| " shafts | 11.6 " " | | |
| | 920 at 130 per cubic foot | | 3 229 200 |
| Iron | | | 9 968 |
| Cement, 678½ barrels at 290 | | | 196 765 |
| Sand, 897½ " 375 | | | 336 562 |
| Total | | | 7 866 059 |
| Displacement, 15.942 x 4.828 x 29.7. | 36 433.3 cubic feet. | | |
| First footing course | 1 802.4 " | | |
| $\frac{111}{200}$ of second footing course | 887.9 " | | |
| | 38 123.6 at 62.33 | | 2 376 244 |
| Net (pounds) | | | 5 489 815 |

| | |
|---|---|
| Area | 1 226.71 square feet. |
| Pressure per square foot | 4 475.2 pounds. |
| " " " inch | 31.08 " |

Superstructure and load:

| | |
|---|---|
| 366 foot truss, dead load, 2 585 pounds per foot | 946 110 |
| Turn-table, etc | 124 190 |
| $\frac{5}{8}$ of total load, 3 000 pounds per foot + 40 000 excess | 711 250 |
| | 1 781 550 |

1 452 pounds per square foot; 10.08 per square inch.
Total, 5 927.2 pounds per square foot; 41.16 per square inch.

## Pier 5.

It is built on a caisson sunk to bed rock. The caisson is similar to that of the other piers, except that the short braces in the crib are arranged differently, four being put in one course and three in the next course above, so as to break joints.

No difficulties were encountered in sinking this caisson.

The pier is similar in all respects to Pier 3, being one foot wider than the other piers.

| | |
|---|---|
| Began building caisson (1885) | July 23d |
| Launched " | September 5th, 11 A.M. |
| Located " | October 3d, 8 A.M. |
| Began concrete in crib | " 7th, P.M. |
| " sinking | " 8th, 1 P.M. |
| Finished concrete in crib | " 13th. |
| Began masonry | " 14th, 9.45 A.M. |
| Finished sinking | " 18th, A.M. |
| Began concrete in chamber | " 18th, 1 P.M. |
| Finished " " | " 20th, 4.30 A.M. |

### Quantities and Weights.

| | | |
|---|---|---|
| Solid contents of masonry | 478 cubic yards. | |
| 421.85 cubic yards limestone 155 pounds per foot | | 1 765 442 |
| Concrete in chamber | 91.0 | |
| " crib | 370.5 | |
| " shafts | 7.0 | |
| | 468.5, 135 pounds per foot. | 1 644 435 |
| 81 306 feet B. M. timber, at 4½ pounds per foot | | 365 877 |
| Wrought and cast-iron in foundation | | 7 925 |
| Wrought-iron in masonry | | 871 |
| 244 barrels cement, 290 pounds | | 70 760 |
| 348 " sand, 375 pounds | | 130 500 |
| Total | | 3 985 810 |
| Less displacement at low-water | | 1 314 362 |
| Net (pounds) | | 2 671 183 |

Area of foundation 702 square feet.

| | Per Square Foot. | Per Square Inch. |
|---|---|---|
| Pressure on Foundations from Substructure.... | 3 805 | 26.4 |
| " " Superstructure... | 1 339 | 9.3 |
| Total.....(pounds) | 5 144 | 35.7 |

Superstructure is the same as Pier 3.

### Pier 6

Is built on a caisson sunk to bed-rock. The caisson and arrangement of braces in crib are the same as in No. 5.

No difficulty was experienced in sinking this caisson, which completed the pneumatic work on the bridge.

Owing to an error in the quarry, the first footing course in this pier was duplicated and the contractors were allowed to use it, thus making the first footing 4 feet high, and the second 2 feet. The pier is, in other respects, similar to Pier 2.

| | |
|---|---|
| Began building caisson (1885) | September 5th, 4 P.M. |
| Launched caisson | October 15th, 1.30 P.M. |
| Located caisson | October 16th, 11 A.M. |
| Began concrete in crib | October 20th, 10.45 A.M. |
| " sinking | October 21st, 9.30 A.M. |
| Finished concrete in crib | October 26th |
| Began masonry | October 27th, 8.10 A.M. |
| Finished sinking | October 31st |
| Began concrete in chamber | October 31st, 11.10 A.M. |
| Finished concrete in chamber | November 1st, 11.15 P.M. |
| " masonry | November 21st, 1.15 P.M. |

### Quantities and Weights.

| | |
|---|---|
| Total contents of masonry..... 442.2 cubic yards. | |
| 375.2 cubic yards limestone, 155 pounds per foot..... | 1 570 212 |
| Concrete in chamber. 91.0 cubic yards. | |
| " " crib.....350.7 " " | |
| " " shafts... 7.7 " " | |
| 449.4, 130 pounds per cubic foot, | 1 577 394 |
| 78 959 feet B. M. timber, 4½ pounds per foot..... | 355 315 |
| Wrought and cast-iron in foundations..... | 7 651 |
| Wrought-iron in masonry..... | 851 |
| 250 barrels cement, 290 pounds..... | 72 500 |
| 355½ barrels sand, 375 pounds..... | 133 312 |
| Total..... | 3 717 235 |
| Less displacement at low water..... | 1 297 754 |
| Net..... | 2 419 481 |

Area of foundation..... 702 square feet

Pressure on foundations. 3 446.5 per square foot, 23.9 per square inch from substructure.

Superstructure:

| | |
|---|---|
| 162 feet span, dead load, per foot, 2 150.. ..... | 348 300 |
| " " live load, per foot, 3 000 + 40 000..... | 526 000 |
| 252 feet 9 inches span, dead load, per foot, 2 536..... | 640,974 |
| " " " live load, per foot, 3 000+40 000, | 798 250 |
| | 2 313 524 |
| One-half of which goes to this Pier=..... | 1 156 762 |

1 648 per square foot, 11.44 per square inch.

Total.....(pounds) 5 094.5 per square foot.
35.34 per square inch.

PIER 7

Is built on a grillage supported by eighty-four piles driven to bed-rock.

A crib 22 x 46 outside, made of twelve courses of 12 x 12 inch timber, each course stepped in ½-inch smaller than the course below, was sunk to 6 feet below low water (the level of the sand was low water). No difficulty was encountered in the sinking (which was done by means of a Joliet pump 8 inches in diameter, and by shoveling), except a delay from high water.

The eighty-four piles were driven in seven rows of twelve piles each, 2 feet 10 inches apart, center to center, in the direction of the bridge, and 3 feet 6½ inches apart at right angles to the same. These piles were driven to refusal, and after being cut off 2 feet below low water mark, made an average of 30.1 feet long.

The space between the piles, from 1½ to 3 feet deep, was then filled with concrete, made 1 cement, 2 sand and 3 broken stone, and the piles capped in the direction of the bridge, and the concrete brought up level with the top of the caps. Next a grillage 18 x 40 feet x 12 inches thick was laid on these caps and drift-bolted at every intersection, and on this the masonry was commenced, consisting of three footing courses each 2 feet high, with 9-inch offsets; on this the neat work begins 9 feet 4 inches wide and 22 feet long between shoulders, with semicircular ends, and is similar to Pier No. 2.

| | |
|---|---|
| Began framing crib (1885)..... | August 20th |
| Located crib..... | " 29th |
| Began driving piles..... | September 26th |
| Finished driving piles..... | October 4th, 3 P.M. |
| Piles sawed off..... | October 5th, 5.45 P.M. |
| Began concrete..... | October 6th, 2 P.M. |
| Finished concrete and began grillage..... | October 8th, 10.30 A.M. |
| Began masonry..... | October 9th, A.M. |
| Finished masonry..... | November 9th, noon |

QUANTITIES AND WEIGHTS.

| | | |
|---|---|---|
| Total contents of masonry | 385.7 cubic yards | |
| Timber in crib | 29 764 feet B. M. | |
| " in grillage | 11 233 " " | |
| Iron in crib | 960 pounds | |
| " in grillage | 1 575 " | |
| Piles | 2 528 lineal feet. | |
| Iron in masonry | 686 pounds. | |
| Concrete | 89.2 cubic yards. | |
| Excavation under water | 225 " " | |
| 336.6 cubic yards of limestone, 155 pounds per foot | | 1 408 671 |
| 214½ barrels cement, 290 | | 62 205 |
| 302½ " sand, 375 | | 113,437 |
| 11 233 feet B. M. timber, 4½ pounds | | 50 542 |
| Iron | | 2 261 |
| 19 488 pounds or 9.744 tons per pile | | 1 637 116 |
| From superstructure and load: | | |
| 162 feet truss, dead load, 2 150 pounds per foot | | 348 300 |
| " " live load, 3 000 + 40 000 | | 526 000 |
| | | 874 300 |

=10 408 pounds or 5.204 tons per pile.
Total, 29 896 pounds or 14.948 tons per pile.

PIER 8

Is built on a grillage supported by eighty-four piles driven to bed-rock.

A crib 22 x 46 outside, made of twelve courses of 12 x 12 inch timber, each course being stepped in one-half inch smaller than the course below it, was sunk to 6 feet below low water. No difficulty was experienced in sinking it, probably on account of the crib having a batter, and the absence of sheet piling, which was not the case with No. 9.

Sinking the crib was begun August 14th, and completed the 19th. The eighty-four piles were driven in seven rows of twelve each, 2 feet 10 inches apart, center to center, in the direction of the bridge, and 3 feet 6½ inches apart at right angles to the same.

These piles were driven to refusal, and after being cut off 2 feet below low water mark, are an average of 27.61 feet long.

The space between the piles, from 1½ to 3 feet deep, was then filled with concrete made—1 cement, 2 sand, and 3 broken stone; the piles were then capped in the direction of the bridge, and the concrete brought up level with the top of the caps; next a grillage 18 x 40 feet was laid on these caps, drift bolted at every intersection, and on this the masonry was commenced, consisting of three footing courses, each 2 feet high, with 9-inch offsets. On this the neat work begins, 9 feet 4 inches wide, 22 feet between shoulders, with semicircular ends, and is carried up similar to the other piers.

| | | |
|---|---|---|
| Began crib (1885) | | July 30th. |
| " sinking | | August 14th. |
| Finished sinking | | " 19th. |
| Began driving piles | | " 20th. |
| Finished driving piles | | " 27th. |
| " grillage | | " 30th. |
| Began masonry | | " 31st. |
| Finished masonry | | September 30th. |
| Total contents of masonry | 386.7 | cubic yards. |
| Timber in crib | 29 764 | feet B. M. |
| " grillage | 11 233 | " |
| Iron in crib | 960 | pounds. |
| " grillage | 1 575 | " |
| Piles | 2 319 | linear feet. |
| Concrete | 93.7 | cubic yards. |
| Iron in masonry | 843 | pounds. |
| Excavation above water | 80 | cubic yards. |
| " under " | 412 | " |

WEIGHTS.

| | |
|---|---|
| 338.4 cubic yards stone, 155 pounds per cubic foot | 1 416 204 |
| 201 barrels cement, 290 pounds each | 58 290 |
| 304 " sand, 375 " | 114 000 |
| 11,233 feet timber, 4½ " | 50 542 |
| Iron | 2 418 |
| Total | 1 641 454 |

19 541 pounds per pile, or ..... 9.770 tons.
Superstructure same as Pier 7,
10 408 pounds per pile, or ..... 5.204 tons.
Total, 29 949 pounds, or ..... 14.974 tons per pile.

PIER 9

Is built on a grillage supported by piles driven to rock.

A crib 22 x 46 outside, made of twelve courses of 12 x 12 pine timber, was sunk to a depth of 5 feet 4 inches below low water.

In this crib eighty-four piles were driven in seven rows of twelve piles each, distant apart, center to center, 2 feet 10 inches in the direction of the bridge, and 3 feet 6½ inches at right angles to the same.

These piles were cut off at the level of 2 feet below low water, and capped with 12 x 12 pine in the direction of the bridge.

The pit was then filled with concrete made—1 cement, 2 sand, and 3 broken stone, up to the top of the caps.

A solid floor of 12 x 12 pine was then laid on these caps, 18 x 40 feet, and drift bolted at every intersection, and on this floor the masonry was commenced (at low water mark).

After beginning the masonry the space between the grillage and the crib was filled with concrete.

The masonry consists of three footing courses, respectively 2 feet 3 inches, 2 feet 3 inches and 2 feet 2 inches, with 9-inch offsets. On these the neat work begins, 9 feet 3 inches wide, 22 feet between

shoulders, with semicircular ends, and similar in all respects to the other fixed piers.

A good deal of trouble was encountered in putting in the foundation of this pier. The sand being at a level of about 12 feet above low water, the work of excavating the pit down to water level was begun April 18th, and had to be stopped April 22d, on account of high water.

Nothing more was done until June 6th, when excavation was again started, and the coffer dam, or crib, put in place on the 12th.

A row of close piles, 3 x 12, was then driven around the crib, about one foot from it, for the purpose of preventing the sand from running in. These piles were all driven on the 17th, and on the 27th the work was again stopped by high water, and resumed on July 2d, when an additional row of close piles, 6 x 12, was begun around the crib. The work was again stopped July 5th by high water, and nothing done until July 21st, when (the water being low enough) the sheet piling was completed, and the work of sinking resumed on the 23d.

On August 1st, the bottom of the crib being then 5 feet 4 inches below low water, the pump used (a 6-inch Bush pump) broke down, and to save time it was decided not to sink the crib any further, as it only had 8 inches more to sink to comply with the specifications.

August 2d driving piles began, and was completed August 10th. The piles were driven to refusal, and the average length, after cutting off, is 26.31 feet.

The rock varied in height about 2 feet.

In sinking the crib the sand was removed by shovels and hoisting; the pump being used to keep the water down.

Subsequent experience on Pier 8 would lead to the belief that this crib could have been sunk in half the time had no sheet piles been driven, and the crib built with a cutting edge and battered sides.

| | |
|---|---|
| Began excavating pit (1885) | April 18th. |
| Stopped by high water | " 22d. |
| Began again | June 6th. |
| " crib | " 12th. |
| " sheet piling (3 inches) | " 15th. |
| Finished " | " 17th. |
| Stopped by high water | " 27th. |
| Began sheet piling (6 inches) | July 2d. |
| Stopped by high water | " 5th |
| Began again (sheet piles) | " 21st. |
| Resumed sinking crib | " 23d. |
| Finished sinking crib | August 1st. |
| Began driving piles | " 2d. |
| Finished driving piles | " 10th. |
| Began cutting off piles | " 10th. |
| Grillage finished | " 13th. |
| Began masonry | " 14th. |
| Finished masonry | " 29th. |
| Total contents of masonry | 378.5 cubic yards. |

| | | |
|---|---|---|
| Timber in crib | 29 764 | feet B. M. |
| " grillage | 11 233 | " |
| Iron in crib | 960 | pounds. |
| " grillage | 1 575 | " |
| Iron in masonry | 636 | " |
| Piles | 2 210 | lineal feet. |
| Iron in masonry | 636 | pounds. |
| Concrete | 136 | cubic yards. |
| Excavation above water | 200 | " |
| " under " | 480 | " |

QUANTITIES AND WEIGHTS.

| | |
|---|---|
| 303.2 cubic yards stone, 155 pounds per foot | 1 268 892 |
| 197½ barrels cement, 290 pounds each | 57 200 |
| 277½ barrels sand, 375 pounds each | 103 969 |
| 11 233 feet B. M. timber, 4½ pounds | 50 544 |
| Iron | 2 211 |
| Total | 1 482 816 |

= 17 652 pounds, or 8.826 tons per pile.

Superstructure same as in Pier 8.

10 408 pounds, or 5.204 tons per pile.

Total.....28 060 pounds, or 14.030 tons per pile.

PIER 10 OR SOUTH ABUTMENT

Is built on a concrete foundation. The level of the ground at the pier was at an average elevation of 397.

A pit was excavated 15 x 31 at bottom, and 4 feet in height of concrete placed on it; the bottom being at an elevation of 376.17, and on this the masonry was commenced May 21st, 8.35 A.M.

The masonry consists of three footing courses, each 2 feet, with 9-inch offsets, and on these the masonry begins, 24 feet x 8 feet 1 inch, and is carried up plumb on back and ends, and with a batter of 2 inches per foot on face to the under side of the coping; finished 24 feet x 5 feet 9 inches.

The coping is 16 inches thick and 25 feet x 6 feet 3 inches.

The ballast wall is 24 feet x 2 feet 6 inches, and 5 feet 4 inches high.

All the masonry, except coping and ballast wall, which are of Eureka Springs limestone, is of sand-stone from a quarry near Mountainburg, Arkansas.

| | | |
|---|---|---|
| Began pit (1885) | May | 9th. |
| Finished pit | " | 16th. |
| Began concrete | " | 18th. |
| Finished concrete | " | 20th. |
| Began masonry | " | 21st, 8.35 A. M.* |
| Finished masonry | June | 9th. |

Total contents of masonry.....173.4 cubic yards.

*First stone of bridge.

| | | |
|---|---|---|
| 68.8 | cubic yards concrete, 130 pounds per foot..... | 241 488 |
| 112 | barrels cement and 124 barrels sand, viz.: | |
| 100 | barrels mixed 1 to 1, and 12 barrels mixed 2 to 1... | |
| 19.6 | cubic yards limestone, 155 pounds per foot..... | 82 026 |
| 136.8 | " sandstone, 139 " " ..... | 513 410 |
| 66 | barrels cement, 290 pounds each..... | 19 140 |
| 114 | " sand, 375 " ..... | 42 750 |
| | Total..... | 898 814 |

Area of foundation..... 465 square feet.

Pressure from substructure:

1 933 pounds per square foot.....13.4 pounds per square inch.

Superstructure:

162-foot truss—total dead and live load, 874 300, half of which goes to abutment—940 pounds per square foot, or 6.52 pounds per square inch.

Total....2 873 pounds per square foot, or 19.92 per square inch.

### Cement.

The cement used was all from the Western Cement Association of Louisville, Ky., and with the exception of two car loads was the "Speed Mills" brand. Two car loads were obtained put up in paper sacks containing about one bushel each. As this cement seemed of poor quality, and did not come up to the specifications, not having the maker's name on it, no more was used.

The specifications were that the cement should be capable of standing a tensile strain of 40 pounds per square inch when mixed pure, exposed for thirty minutes in air and twenty-four hours under water.

Of 5 000 barrels tested, only twenty-eight failed to stand this test and were accordingly rejected.

The average tensile strength of the 5 000 tested was 88.6 pounds per square inch.

Tests were made by mixing the cement with as little water as possible while making it a coherent mass. Of this a briquette was moulded and exposed thirty minutes in air and twenty-four hours under water, after which it was broken in a Fairbanks testing machine, which was automatic in its action.

For experiment, some other tests of this cement were made, with the following results:

Mixed as above and exposed seven days under water, 1 800 barrels averaged 120.6 pounds.

Also a number of briquettes were kept for a longer period and tested with the following results:

| | | | | | |
|---|---|---|---|---|---|
| 70 | days | under | water..... | 250 | pounds. |
| 100 | " | " | " ..... | 445 | " |
| 107 | " | " | " ..... | 635 | " |
| 122 | " | " | " ..... | 437 | " |
| 160 | " | " | " ..... | 342 | " |

Two different lots of about twelve briquettes each were mixed 1 cement to 2 sand, and after twenty-four hours under water showed a strength of 12.2 and 15 pounds respectively.

A lot mixed 1 cement to 1 sand after twenty-four hours under water showed a strength of 42 pounds.

A lot mixed 1 cement to 3 sand after twenty-four hours under water showed a strength of 10 pounds.

Also a lot mixed 1 cement to 2 sand and exposed seven days under water showed a strength of 20.4 pounds.

On adding up the various amounts used in the work, and adding what was left over, and what was known to be wasted, only eight barrels are unaccounted for.

A total of 5 892 barrels was used in the work, which was distributed as below :

| | Working Chambers. | | | Cribs. | | | Air Shafts. | | |
|---|---|---|---|---|---|---|---|---|---|
| Pier. | Cubic Yards Concrete. | Barrels Cement. | Barrels per Cubic yard. | Cubic Yards Concrete. | Barrels of Cement. | Barrels per Cubic Yard. | Cubic Yards Concrete. | Barrels Cement. | Barrels per Cubic Yard. |
| 1 | 68.3 | 105 | 1.53 | 66.3 | 63 | 0.92 | 11.9 | 12 | 1.0 |
| 2 | 103 | 138 | 1.34 | 144 | 195 | 1.35 | 14.2 | 21 | 1.48 |
| 3 | 91 | 93 | 1.02 | 334.4 | 297 | 0.89 | 2.7 | 2½ | 0.9 |
| 4 | 198.4 | 198 | 1.0 | 710 | 725 | 1.02 | 11.6 | 17 | 1.46 |
| 5 | 91 | 90 | 0.99 | 370.5 | 370½ | 1.0 | 7 | 6½ | 0.92 |
| 6 | 91 | 95½ | 1.05 | 350.7 | 331 | 0.94 | 8 | 5 | 0.62 |
| 7 | no chamber | .... | .... | 89.2 | 140 | 1.57 | .... | .... | .... |
| 8 | " " | .... | .... | 93.7 | 147 | 1.56 | .... | .... | .... |
| 9 | " " | .... | .... | 136 | 159½ | 1.16 | .... | .... | .... |
| 10 | " " | .... | .... | 68.8 | 112 | 1.63 | .... | .... | .... |
| Total... | 642.7 | 719½ | 1.12 | 2,363.6 | 2,540 | 1.07 | 55.4 | 64 | 1.15 |

## Plant Used.

By William Sooysmith & Son, Contractors for Substructure :

| | | | |
|---|---|---|---|
| Barge for pressure machinery | | 22 x 56, 3 feet deep. | |
| " | concrete mixer | 20 x 50, 3 | " |
| " | stone, sand, etc. | 1 | 16 x 50, 3 " |
| " | " | 1 | 18 x 50, 3 " |
| " | " | 2 | 16 x 44, 2½ " |
| " | " | 1 | 12 x 48, 2½ " |
| " | derrick | 1 | 20 x 30, 2 " |
| " | coal | 1 | 10 x 30, 2 " |

Pressure machinery consisted of one double and one single Ingersoll Air Compressor with one 50 and one 60 horse-power boiler. On same boat was a double Worthington pump for jet.

1 double engine to run concrete mixer.
1 hoisting engine on derrick barge, and
1 Foster rock crusher and engine.

By Masonry Department, Union Bridge Company:
2 barges 26 x 55, 4 feet deep, each with derrick and hoisting engine, for handling stone.
3 barges 20 x 50, 3 feet deep, for sand and cement.
4 derricks in stone yard worked by 2 hoisting engines.
By Union Bridge Company, for foundations and erecting:
1 barge 22 x 55, 3 feet deep, for pile driver and jet pump.
1 " 22 x 50, 3 " timber, etc.
2 small coal barges.
A number of skiffs.
1 large hoisting engine for land pile driver.
1 hoisting engine (detached), with horizontal boiler on boat for pile driver and Cameron pump.
2 hoisting engines.
1 9-inch Joliet pump.
1 6-inch Bush pump.
1 double Cameron force pump (on pile driver boat).
3 pile drivers (land), one with 4 700-pound hammer.
2 derricks for unloading and 1 (on trestle) for loading iron.

FINAL ESTIMATE FOR SINKING CAISSONS.

| | Original Elevation. | Actual Elevation. | Difference + or −. |
|---|---|---|---|
| Pier 1 | 350 | 351.15 | + 1.15 |
| " 2 | 342 | 338.47 | − 3.53 |
| " 3 | 332 | 330.23 | − 1.77 |
| " 4 | 330 | 329.86 | − 0.14 |
| " 5 | 328 | 331.24 | + 3.24 |
| " 6 | 330 | 330.90 | + 0.90 |

| | |
|---|---|
| Difference in favor of Union Bridge Company in Piers 1, 2, 3, 5 and 6, 0.01 foot at $400 per foot, | $ 4 00 |
| Pier 4, 0.14 feet at $600 per foot | 84 00 |
| | $88 00 |

TRIANGULATION AND LOCATION OF PIERS.

A base line was measured on the south bank of the river, both up and down stream from axis of bridge, and a series of triangles made to a point on the north bank. When the distance had been determined as closely as was possible (instrument reading to 20 seconds) by doubling and trebling the angles, the distance from the base line to Pier 10 was laid off on each base line, and from those points the several distances of the piers were carefully measured out both up and down stream, and points set. At these points angles of 45 degrees were turned and reference points put in on north bank of river, so that in locating a pier three lines were available as a check on each other, viz., the center line and two lines at an angle of 45 degrees to it.

It was found in locating the piers that these lines agreed very closely (within ½ inch), except on Pier 2, where there was a difference of 3½ inches. This was then checked by a wire from Pier 1, and the line

which agreed most nearly with the wire used as correct. On the pier being finished and the false work erected this distance was found exact.

The caissons were located by first locating the pile-driver boat, and driving two piles at each corner of the place where the caisson should be, and 4 feet distant from the caisson. When the caisson was floated into position it was moored to these piles, and the center line being marked on it, was brought into exact position, as defined, by three transits on shore; one on the axis of the bridge and the other two on reference points up and down stream. Afterwards, during sinking and building masonry, only one base line was used until the masonry was finished, when the center for the span was given by three transits, as at first.

In measuring the base line a steel wire made for the purpose and 300 feet long was used. First a length of 300 feet was measured very carefully and repeatedly with a standard 50-foot steel tape, and the 300-foot wire stretched over this distance with a spring balance at the end; the amount of pull required to make the 300 feet being noted and afterwards used in measuring the base lines.

In pile foundation concrete is around heads of piles and in grillage.

| | | | | | | |
|---|---|---|---|---|---|---|
| Pile foundation, 1 cement, 2 sand, 3 stone | | | | | Pier | 7 |
| " | " | " | " | ..... | " | 8 |
| " | " | " | " | ..... | " | 9 |
| 100 barrels (in sacks), mixed 1 to 1; rest, 2 to 1 | | | | ..... | " | 10 |

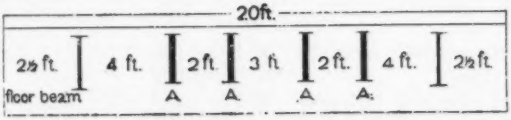
CEMENT IN MASONRY.

| PIER. | Total Yards. | Barrels Cement. | Barrels per Cubic Yard. |
|---|---|---|---|
| 1 | 379.8 | 222 | 0.58 |
| 2 | 436.2 | 230½ | 0.54 |
| 3 | 491 | 264½ | 0.54 |
| 4 | 987.4 | 678½ | 0.70 |
| 5 | 471 | 244 | 0.51 |
| 6 | 434.2 | 250 | 0.57 |
| 7 | 385.7 | 214½ | 0.55 |
| 8 | 386.7 | 201 | 0.52 |
| 9 | 378.5 | 197½ | 0.57 |
| 10 | 173.4 | 66 | 0.36 |
| | 4,523.9 | 2,568½ | 0.57 |

Beds and joints of masonry laid in mortar made of 1 cement to 2 sand; grout mixed 1 to 1.

SPECIFICATIONS FOR SUPERSTRUCTURE.

*General Description.*—The Bridge shall consist of one draw or pivot span of 370 feet from centers of piers supporting the adjoining fixed spans, three fixed spans of 256 feet each from center to center of piers and four fixed spans of 165 feet each from center to center of piers. The spans shall be arranged as directed by the Chief Engineer of the St. Louis and San Francisco Railway, and shall be designed to permit flooring over the cross-ties to fit it for a highway as well as a railway bridge. The stringers to support the track and proposed floor shall be arranged approximately as below, there being four lines of main track stringers and suitable supports riveted to the floor-beams for two lines of auxiliary stringers.




The draw or pivot span shall be constructed on latest and most approved designs. The turn-table shall be rim-bearing. All cast-iron surfaces of beams, rim and track shall be planed. The circular girder will be of wrought-iron, 27 feet in diameter. Wheels shall be at least 18 inches in diameter and 8 inches tread, and turned to uniform size and made of the best car-wheel iron. The turning gear, elevating screws and connecting shafting shall be of the best class of finished machine work. The latches shall be of the usual character, and, as well as the elevating screws, will be worked from the pivot pier. Hand power will be used to work the machinery and suitable gearing shall be provided.

Open-hearth steel of quality and strength as hereinafter specified may be used in the top and bottom chords, end posts, main ties and pins of the three 256-foot spans, and the draw span. The allowable strains per square inch for steel so used shall be:

1. In compression—50 per cent. above strain allowed for wrought-iron.
In tension—40 per cent. above strain allowed for wrought iron.
In shearing—30 per cent. above strain allowed for wrought-iron.

2. In calculating strains, the length of span shall be understood to be the distance between centers of end pins for trusses, and between centers of bearing plates for all beams and girders.

3. There shall be a clear head-room of 20 feet above the base of the rails, and clear width of 18 feet. Head-room.

4. The wooden floor will consist of transverse floor timbers, extending the full width of the bridge, supporting the rails and guard beams. Their scantling will vary with circumstances. They will be furnished and put on by the Railroad Company. Floor.

5. Spans shall be proportioned to carry the following loads: Dead load.

*First.*—The weight of iron or steel in the structure.

*Second.*—A floor weighing 950 pounds per lineal foot of track, to consist of the rails, ties and guard timbers only.

These two items, taken together, shall constitute the "dead load."

Live loads. The live load shall be as follows:

For chord strains of trusses—3 000 pounds per lineal foot of bridge.

For web strains of trusses—5 000 pounds per lineal foot for a length of 20 feet and 3 000 pounds per lineal foot on remainder of span.

For auxiliary stringers—800 pounds per lineal foot.

For main track stringers—(16-foot panel) 6 400 pounds per lineal foot.

For main track stringers—(18-foot panel) 6 300 pounds per lineal foot.

For main track stringers—(20-foot panel) 6 200 pounds per lineal foot.

For main track stringers—(22-foot panel) 6 100 pounds per lineal foot.

For main track stringers—(24-foot panel) 6 000 pounds per lineal foot.

For floor-beams—(16-foot panel) 96 000 pounds, distributed at AAAA on diagram.

For floor-beams—(18-foot panel) 102 000 pounds, distributed at AAAA on diagram.

For floor-beams—(20-foot panel) 108 000 pounds, distributed at AAAA on diagram.

For floor-beams—(22-foot panel) 114 000 pounds, distributed at AAAA on diagram.

For floor-beams—(24-foot panel) 120 000 pounds, distributed at AAAA on diagram.

Stresses. The maximum strains due to all positions of the above "live load," and of the "dead load," shall be taken to proportion all the parts of the structure.

Lateral Stresses. 6. To provide for wind strains and vibrations, the top lateral bracing in deck bridges, and the bottom lateral bracing in through bridges, shall be proportioned to resist a lateral force of 500 pounds for each foot of the span; 300 pounds of this to be treated as a moving load.

The bottom lateral bracing in deck bridges, and the top lateral bracing in through bridges, shall be proportioned to resist a lateral force of 200 pounds for each foot of the span.

Temperature. 7. Variations in temperature, to the extent of 150 degrees, shall be provided for.

8. All parts shall be so designed that the strains coming upon them can be accurately calculated.

Plans and Strain Sheets.

9. Strain sheets and a general plan, showing the dimensions of the parts and general details, must accompany each proposal.

10. Upon the acceptance of a proposal, a full set of working drawings must be submitted for approval by the Chief Engineer of the Railroad Company, before the work is commenced.

Form of Truss.

11. Unless otherwise specified, the form of truss may be selected by the builder; but to secure uniformity in appearance, it is desired that all "through" trusses shall be built with inclined end-posts.

12. In comparing competitive plans, the relative cost of the wooden floors required will be taken into consideration.

13. The following clauses are all intended to apply to iron construction. Parties proposing to substitute steel for particular parts will be required to furnish evidence of its strength, elasticity, uniformity in production and adaptability to the intended purpose.

## Proportion of Parts.

Tensile Strains.

1. All parts of the structure shall be so proportioned that the maximum strains produced shall in no case cause a greater tension than the following:

| | Pounds per Square Inch. |
|---|---|
| On lateral bracing | 15 000 |
| " solid rolled beams, used as cross floor-beams and stringers. | 10 000 |
| " bottom chords and main diagonals | 10 000 |
| " counter rods and long verticals | 8 000 |
| " bottom flange of riveted cross-girders, net section | 8 000 |
| " bottom flange of riveted longitudinal plate girders over 20 feet long, net section | 8 000 |
| " bottom flange of riveted longitudinal plate girders under 20 feet long, net section | 7 000 |
| " floor-beam hangers, and other similar members liable to sudden loading | 6 000 |

Compressive Strains.

2. Compression members shall be so proportioned that the maximum load shall in no case cause a greater strain than that determined by the following formulas:

$$P = \frac{8\,000}{1 + \frac{L^2}{40\,000\,R^2}}$$ for square end compression members.

$$P = \frac{8\,000}{1 + \frac{L^2}{30\,000\,R^2}}$$ for compression members with one pin and one square end.

$$P = \frac{8\,000}{1 + \frac{L^2}{20\,000\,R^2}}$$ for compression members with pin bearings.

$P$ = the allowed compression per square inch of cross-section.
$L$ = the length of compression member in inches.
$R$ = the least radius of gyration of the section in inches.

3. The lateral struts shall be proportioned by the above formulas to resist the resultant due to an assumed initial strain of 10 000 pounds per square inch upon all the rods attaching to them, produced by adjusting the bridge.

4. In beams and girders, compression shall be limited as follows:

| | Pounds per Square Inch. |
|---|---|
| In rolled beams, used as cross floor-beams and stringers..... | 10 000 |
| " riveted plate girders, used as floor beams, gross section.... | 6 000 |
| " " longitudinal plate girders over 20 feet long, gross section..... | 6 000 |
| " riveted longitudinal plate girders under 20 feet long, gross section..... | 5 000 |

5. Members subjected to alternate strains of tension and compression shall be proportioned to resist each of them. The strains, however, shall be assumed to be increased by an amount equal to $\frac{8}{10}$ of the least strain.

Shearing Strains.

6. The rivets and bolts connecting all parts of the girders must be so spaced that the shearing strain per square inch shall not exceed 6 000 pounds, nor the pressure upon the bearing surface exceed 12 000 pounds per square inch of the projected semi-intrados (diameter × thickness of piece) of the rivet or bolt hole.

Bending Strains.

7. Pins shall be so proportioned that the shearing strain shall not exceed 7 500 pounds per square inch; nor the crushing strain upon the projected area of the semi-intrados (diameter × thickness of piece) of any member connected to the pin be greater than 12 000 pounds per square inch; nor the bending strain exceed 15 000 pounds per square inch, when the centers of bearings of the strained members are taken as the points of application of the strains.

8. In case any member is subjected to a bending strain from local loadings (such as distributing floors on deck bridges), in addition to the strain produced by its position as a member of the structure, it must be proportioned to resist the combined strains.

Plate Girders.

9. Plate girders shall be proportioned upon the supposition that the bending or chord strains are resisted entirely by the upper and lower flanges; and that the shearing or web strains are resisted entirely by the web-plate.

10. The compression flanges of beams and girders shall be stayed against transverse crippling, when their length is more than thirty times their width.

11. The unsupported width of any plate subjected to compression shall never exceed thirty times its thickness.

12. In members subject to tensile strains, full allowance shall be made for reduction of section by rivet-holes, screw-threads, etc.

13. The iron in the web-plates shall not have a shearing strain greater than 4 000 pounds per square inch, and no web-plate shall be less than $\frac{1}{4}$ inch in thickness.

14. No wrought iron shall be used less than $\frac{5}{16}$ inch thick, except in places where both sides are always accessible for cleaning and painting.

### Details of Construction.

1. All the connections and details of the several parts of the structure shall be of such strength that upon testing, rupture shall occur in the body of the members rather than in any of their details or connections.

2. Preference will be had for such details as will be most accessible for inspection, cleaning and painting.

3. The web of plate girders must be spliced at all joints by a plate on each side of the web. T-iron must not be used for splices.

4. When the least thickness of the web is less than $\frac{1}{80}$ of the depth of a girder, the web shall be stiffened at intervals not over twice the depth of the girder.

5. The pitch of rivets in all classes of work shall never exceed 6 inches, nor sixteen times the thinnest outside plate, nor be less than three diameters of the rivet.

6. The rivets used will generally be $\frac{3}{4}$ and $\frac{7}{8}$ inch diameter.

7. The distance between the edge of any piece and the center of a rivet-hole must never be less than $1\frac{1}{4}$ inches, except for bars less than $2\frac{1}{2}$ inches wide; when practicable it shall be at least two diameters of the rivet.

8. When plates more than 12 inches wide are used in the flanges of plate or lattice girders, an extra line of rivets, with a pitch of not over 9 inches, shall be driven along each edge to draw the plates together, and prevent the entrance of water.

9. In punching plate or other iron, the diameter of the die shall in no case exceed the diameter of the punch by more than $\frac{1}{16}$ of an inch.

10. All rivet-holes must be so accurately punched that when the several parts forming one member are assembled together, a rivet $\frac{1}{16}$ inch less in diameter than the hole can be entered, hot, into any hole, without reaming or straining the iron by "drifts."

11. The rivets, when driven, must completely fill the holes.

12. The rivet heads must be hemispherical, and of a uniform size for the same sized rivets throughout the work. They must be full and neatly made, and be concentric to the rivet-hole.

13. Whenever possible, all rivets must be machine driven.

14. The several pieces forming one built member must fit closely together, and when riveted shall be free from twists, bends or open joints.

15. All joints in riveted work, whether in tension or compression members, must be fully spliced, as no reliance will be placed upon abutting joints. The ends, however, must be dressed straight and true, so that there shall be no open joints.

Lower Chords and Suspension Bars.

16. The heads of eye-bars shall be so proportioned that the bar will break in the body instead of in the eye. The form of the head and the mode of manufacture shall be subject to the approval of the chief engineer of the railroad company.

17. The bars must be free from flaws, and of full thickness in the necks. They shall be perfectly straight before boring. The holes shall be in the center of the head and on the center line of the bar.

18. The bars must be bored of exact length, and the pin hole $\frac{1}{50}$ inch larger than the diameter of the pin.

19. The lower chord shall be packed as narrow as possible.

Pins.

20. The pins shall be turned straight and smooth, and shall fit the pin holes within $\frac{1}{50}$ of an inch.

21. The diameter of the pin shall not be less than two-thirds the largest dimension of any tension member attached to it. Its effective length shall not be greater than the breadth of the foot of the post, plus four times the diameter of the pin. The several members attaching to the pin shall be packed close together, and all vacant spaces between the chords and posts must be filled with wrought-iron filling rings.

Upset Screw Ends.

22. All rods and hangers with screw ends shall be upset at the ends, so that the diameter at the bottom of the threads shall be $\frac{1}{10}$ inch larger than any part of the body of the bar.

23. All threads must be of the United States standard, except at the ends of the pins.

Floor Beam Hangers.

24. Floor beam hangers shall be so placed that they can be readily examined at all times. When fitted with screw ends they shall be provided with check nuts.

25. When bent loops are used, they must fit perfectly around the pin throughout its semi-circumference.

Compression Members.

26. Compression members shall be of wrought-iron or open hearth steel of approved forms.

27. The pitch of rivets for a length of two diameters at the ends shall not be over four times the diameter of the rivets.

28. The open sides of all trough-shaped sections shall be stayed by diagonal lattice work, at distances not exceeding the width of the member. The size of bars shall be duly proportioned to the width.

29. All pin-holes shall be reinforced by additional material so as not to exceed the allowed pressure on the pins. These re-inforcing plates must contain enough rivets to transfer the proportion of pressure which comes upon them.

30. Pin-holes shall be bored exactly perpendicular to a vertical plane, passing through the center line of each member, when placed in a position similar to that it is to occupy in the finished structure.

31. The ends of all square-ended members shall be planed smooth, and exactly square to the center line of strain. Abutting Joints.

32. All members must be free from twists or bends. Portions exposed to view shall be neatly finished.

33. The sections of the top chord shall be connected at the abutting ends by splices sufficient to hold them truly in position. Splicing of Top Chords.

34. In no case shall any lateral or diagonal rod have a less area than $\frac{3}{4}$ of a square inch. Lateral Bracing.

35. The attachment of the lateral system to the chords shall be thoroughly efficient. If connected to suspended floor-beams, the latter shall be stayed against all motion.

36. All through bridges with top lateral bracing shall have wrought-iron portals, of approved design, at each end of the span, connected rigidly to the end posts.

37. When the height of the trusses exceeds 25 feet, overhead diagonal bracing shall be attached to each post and to the top lateral struts. Transverse Diagonal Bracing.

38. All bed plates must be of such dimensions that the greatest pressure upon the masonry shall not exceed 250 pounds per square inch. Bed Plates.

39. All spans shall have at one end nests of turned friction rollers, formed of wrought-iron, running between planed surfaces. The rollers shall not be less than 2 inches diameter, and shall be so proportioned that the pressure per lineal inch of rollers shall not exceed the product of the square root of the diameter of the roller, in inches, multiplied by 500 pounds ($500\sqrt{d}$). Friction Rollers.

40. All spans shall be given a camber, by making the panel lengths of the top chord longer than those of the bottom chord in the proportion of $\frac{1}{8}$ of an inch to every ten feet. Camber.

## Quality of Iron.

1. All wrought-iron must be tough, fibrous and uniform in character. It shall have a limit of elasticity of not less than 26 000 pounds per square inch.

Finished bars must be thoroughly welded during the rolling, and free from injurious seams, blisters, buckles, cinder spots or imperfect edges.

2. For all tension members the muck bars shall be rolled into flats, and again cut, piled and rolled into finished sizes. They shall stand the following tests: Tension Tests.

3. Full-sized pieces of flat, round or square iron, not over $4\frac{1}{2}$ inches in sectional area, shall have an ultimate strength of 50 000 pounds per square inch, and stretch $12\frac{1}{2}$ per cent. in their whole length.

Bars of a larger sectional area than $4\frac{1}{2}$ square inches, when tested in the usual way, will be allowed a reduction of 1 000 pounds per square inch for each additional square inch of section down to a minimum of 46 000 pounds per square inch.

4. When tested in specimens of uniform sectional area of at least ½ square inch for a distance of 10 inches, taken from tension members which have been rolled to a section not more than 4½ square inches, the iron shall show an ultimate strength of 52 000 pounds per square inch, and stretch 18 per cent. in a distance of 8 inches.

Specimens taken from bars of a larger cross-section than 4½ inches will be allowed a reduction of 500 pounds for each additional square inch of section, down to a minimum of 50 000 pounds.

5. The same sized specimen taken from angle and other shaped iron shall have an ultimate strength of 50 000 pounds per square inch, and elongate 15 per cent. in 8 inches.

6. The same sized specimen taken from plate iron shall have an ultimate strength of 48 000 pounds, and elongate 15 per cent. in 8 inches.

Bending Tests.

7. All iron for tension members must bend cold for about 90 degrees, to a curve whose diameter is not over twice the thickness of the piece, without cracking. At least one sample in three must bend 180 degrees to this curve without cracking. When nicked on one side and bent by a blow from a sledge, the fracture must be nearly all fibrous, showing but few crystalline specks.

8. Specimens from angle, plate and shaped iron must stand bending cold through 90 degrees and to a curve whose diameter is not over three times its thickness, without cracking.

When nicked or bent, its fracture must be mostly fibrous.

9. Rivets and pins shall be made from the best double refined iron.

Cast-iron Tests.

10. The cast-iron must be of the best quality of soft gray iron.

11. All facilities for inspection of iron and workmanship shall be furnished by the contractor. He shall furnish without charge such specimens (prepared) of the several kinds of iron to be used as may be required to determine their character.

12. Full-sized parts of the structure may be tested at the option of the chief engineer of the railroad company; but if tested to destruction, such material shall be paid for at cost, less its scrap value, to the contractor, if it proves satisfactory. If it does not stand the specified tests it will be considered rejected material, and be solely at the cost of the contractor.

### Quality of Steel.

The steel shall be manufactured by the open-hearth process; Bessemer steel will not be accepted, unless satisfactory to the chief engineer of the railway company. A small ingot shall be cast from every charge, and from this ingot a sample bar ½ of an inch in diameter shall be rolled; if this bar fails to meet the requirements of the laboratory tests, the whole charge shall be rejected.

Steel used in the compression members, bolsters, bearing plates, pins and rollers shall contain not less than $\frac{3}{100}$ nor more than $\frac{4.5}{100}$ of one per

cent. of carbon, and less than $\frac{1}{10}$ of one per cent. of phosphorus. A sample test bar, $\frac{3}{4}$ of an inch in diameter, shall bend 180 degrees around its own diameter without sign of crack or flaw. The same bar tested in a lever machine shall show an elastic limit of not less than 50 000 pounds and an ultimate strength of not less than 80 000 pounds per square inch; it shall elongate at least 15 per cent. in a length of 8 inches before breaking, and shall have a reduced area of 35 per cent. at the point of fracture. It shall be incapable of tempering.

Steel for rivets and eye-bars shall contain not more than $\frac{25}{100}$ of one per cent. of carbon and less than $\frac{1}{10}$ of one per cent. of phosphorus. A sample bar $\frac{3}{4}$ of an inch in diameter shall bend 180 degrees, and be set back upon itself without showing crack or flaw; when tested in a lever machine it shall have an elastic limit of not less than 40 000 pounds, and an ultimate strength of not less than 70 000 pounds per square inch; it shall elongate at least 18 per cent. in a length of 8 inches, and shall show a reduction of at least 45 per cent. at the point of fracture. In full sized bars this steel shall have an elastic limit of at least 35 000 pounds, and an ultimate strength of at least 65 000 pounds per square inch; it shall elongate 10 per cent. before breaking, and for strains less than 30 000 pounds per square inch shall show a modulus of elasticity between 28 000 000 and 30 000 000 pounds.

Facilities for testing the sample bars shall be furnished by the contractor at a point convenient to the steel works, and the tests shall be made at the expense of the contractor and under the direction of the chief engineer.

The steel plates for the chords and end posts shall be rolled in a universal mill.

Steel fo pins shall not be hammered, but rolled between gothic rolls.

The riveted steel work used shall be punched with holes not larger than $\frac{1}{16}$ of an inch in diameter; the several parts of each member shall then be assembled, and the holes shall be reamed to $\frac{1}{16}$ of an inch in diameter, at least $\frac{1}{10}$ of an inch being taken out all around; the sharp edge of the reamed hole shall be trimmed so as to make a slight fillet under the rivet head, and the pieces shall be riveted together without taking apart; all rivets in steel members shall be of steel; they shall be of such size that they will fill the hole before driving, and, whenever possible, shall be driven by power. All bearing surfaces shall be truly faced. The chord pieces shall be fitted together in the shop, in lengths of at least five panels, and marked; when so fitted, there shall be no perceptible wind in the length laid out. The pin-holes shall be bored truly, so as to be equally distant, parallel with one another and at right angles to the axis of the member.

So far as possible, all rivets shall be driven by power. The holes for the rivets connecting the floor-beams with the posts, and with the bolsters, which must be driven after erection, shall be accurately drilled to a

templet; these holes shall be 1 inch in diameter, and the rivets $\frac{1}{16}$ of an inch in diameter before driving. The pin-holes in the vertical post shall be truly parallel with one another and at right angles to the axis of the post.

Power riveters shall be direct acting machines worked by steam, hydraulic pressure or compressed air, and capable of holding on to the rivet when the upsetting is completed. Cam riveters will not be allowed.

The heads of steel eye-bars shall be formed by upsetting and forging into shape, or by such other process as may be accepted by the engineer; no welds will be allowed. After the working is completed, the bars shall be annealed by heating them to a uniform dark red heat throughout their entire length, and allowing them to cool slowly. Four sample bars of sizes required in the work shall first be manufactured by the contractor, and tested under the direction of the engineer; these bars shall meet the requirements above specified, and at least three of them shall break in the body of the bar. If the tests of these four bars are satisfactory, the contractor shall proceed with the manufacture of the full order of steel bars for the work, and from the bars so manufactured the inspector shall, from time to time, select six bars to be tested to breaking, which bars shall also conform to the requirements of the specifications. Should these test bars fail to meet the requirements of the specifications, the whole lot of bars may be rejected. If required by the chief engineer, all finished tension members of steel shall be tested to a strain of 20 000 pounds per square inch before shipment, and all bars showing permanent set or other defect under this test shall be rejected.

### Workmanship.

1. All workmanship shall be first-class in every particular.

2. Abutting joints in the trusses shall be in exact contact throughout.

3. Bars which are to be placed side by side in the structure shall be bored at the same temperature, and of such equal length that, upon being piled on each other, the pins shall pass through the holes at both ends without driving.

4. Whenever necessary for the protection of the thread, provision shall be made for the use of pilot nuts in erection.

### Painting.

1. All work shall be painted at the shop with one good coat of selected iron-ore paint and pure linseed oil.

2. In riveted work, all surfaces coming in contact shall be painted before being riveted together.

Bed plates, the inside of closed sections, and all parts of the work which will not be accessible for painting after erection, shall have two coats of paint.

3. Pins, bored pin-holes and turned friction rollers shall be coated with white lead and tallow before being shipped from the shop.

4. After the structure is erected, the iron work shall be thoroughly and evenly painted with two additional coats of paint, mixed with pure linseed oil, of such color as may be directed, the tension members being, however, generally of lighter color than the compression members.

## Erection.

1. The contractor shall furnish all false work and other staging (the plan and construction of which must be approved by the chief engineer), and shall erect and adjust all the iron work complete.

2. The contractor shall so conduct all his operations as not to impede the running of the trains, or the operations of the road.

3. The contractor shall assume all risks of accidents to men or material during the manufacture and erection of the bridge.

4. The river shall be kept free for navigation during the construction and erection of the bridge, and on completion all temporary work shall be removed, leaving the river entirely unobstructed except by actual space occupied by the masonry.

## Specifications for the Substructure.

The substructure shall include the foundations and masonry of the bridge as shown on the plans and profile furnished by the railway company.

*Location of Piers.*—The piers will be located as shown on plans, unless it should be deemed advisable to move the draw-span further north, in which case there will be two piers north of the pivot-pier and seven piers south of the pivot-pier instead of the three piers north of the pivot-pier and six piers south of it, as shown on plan.

*Designation and Description of Piers.*—The piers are numbered 1, 2, 3, 4, 5, 6, 7, 8, 9 and 10, beginning at the north or Van Buren shore.

Pier No. 1 will be an abutment pier, and will rest on the underlying bed-rock. I. Pier 1.

1. Piers Nos. 2, 3, 4, 5 and 6 will be in the channel of the river, and shall be founded on caissons sunk to the underlying bed-rock. II. Piers 2, 3, 4, 5 and 6.

(*Note.*—In sinking the caissons, the pneumatic process will be preferred, as offering the most satisfactory method of securing the desired result; but other plans, involving the use of open caissons of wood or wrought-iron, will be considered, the railway company reserving the right to reject any or all plans submitted.) Pneumatic process. Caissons.

2. The caissons shall be constructed on plans furnished by the engineer, or upon other suitable plans approved by him.

3. The frames shall be of oak or yellow pine timber. The outside sheathing shall consist of two thicknesses of 3-inch oak plank, the planks of the inside course to be inclined at an angle of 45 degrees.

4. The working-chamber shall be lined with 3-inch oak or yellow pine plank, jointed and calked.

5. Each course of timber in the sides shall be fastened to the course below, with drift bolts not more than 3 feet apart. These drift bolts shall be $\frac{7}{8}$ inch square, and shall reach through two courses of timber and 6 inches into the next course, being generally 30 inches long. The oak sheathing shall be spiked with 7-inch by $\frac{3}{4}$-inch wrought-iron spikes. At least two spikes per square foot shall be used in each course of planking.

6. Cutting edges of wrought iron plate shall be provided if required by the engineer.

7. The crib work above the working-chamber shall be filled with concrete, the surface of which shall be kept above water, necessary openings being left for working and supply shafts.

8. The caissons shall be sunk by the pneumatic process, the sand or other material being removed by suitable apparatus for the purpose.

9. The air pressure shall at all times be maintained in the caisson. Two independent air compressors shall be furnished, each of sufficient capacity to maintain the pressure in the caisson.

10. The position of the caisson shall not at any time vary more than 18 inches from its correct position as determined by the engineer.

11. The caisson shall not vary more than 18 inches from a level in its length or 6 inches in its width while being sunk, and shall be brought to a level when completed.

12. The crib-work and concrete above working-chamber shall be built to within 4 feet below low-water mark as determined by the engineer.

13. A close joint shall be made between the cutting edge of the caissons and the bed-rock, either by cutting away the rock or ramming bags of concrete into the spaces between the cutting edge of the caisson and the rock, as the engineer may direct.

14. All sand, mud or other objectionable material shall be removed from the working-chamber on reaching the rock, and it shall be filled with concrete thoroughly rammed.

15. The masonry shall be begun when the concrete and crib-work is ready for it—the foundation course being at least 4 feet below low water. All shafts left open during the progress of the work shall be filled with concrete, as may be directed by the engineer.

III. Piers 7, 8, 9 and 10.

1. Piers Nos. 7, 8 and 9 will have a pile foundation.

2. An open caisson, 20 by 46 feet and 12 feet high, built of 12 x 12 pine or oak pinned together with oak trenails and planked on the outside with one course of 3-inch oak plank, shall be sunk at least 6 feet below low water by dredging from the inside.

3. In the interior of this crib eighty-four piles shall be driven to the rock, either by water jets or by a hammer of suitable weight and fall.

4. The piles shall be of oak or cypress, not less than 10 inches diameter at the small end, nor less than 16 inches at the large end, and shall be sound and straight. Piles.

5. The piles shall be cut off square and level 4 feet below low water and capped with two courses of 12 x 12 oak. The lower course of timbers shall be fastened to each pile with inch drift-bolts 26 inches long; and the second course fastened to the first course at each intersection with inch drift-bolts 22 inches long.

6. The space around the piles and between the timber crib-work shall be filled with concrete level with the top of the grillage, on which the masonry shall be started.

7. Rip-rap shall be disposed around the foundation as directed by the engineer.

8. Pier 10 will have a concrete foundation as shown on plan.

IV. Concrete.

1. Concrete shall be made of one part of cement, two of sand, and three to five parts of broken stone, as directed by the engineer.

2. It shall be thoroughly mixed and well rammed, especially in the working-chambers.

3. The stone for concrete shall be broken so as to pass through a 2½-inch ring, and shall be entirely free from dirt or other foreign matter.

V. Rip-rap.

Rip-rap shall be disposed about piers or elsewhere, as directed by the engineer. One-third of it shall be stone that will average one-third of a cubic yard in size, and the remainder range in size from ¼ to 2 cubic feet.

VI. Masonry.

1. All masonry shall conform to the dimensions and plans furnished by the engineer.

2. Masonry will be first-class, pitched-draught, rock-face work, built in regular courses.

3. The rock-face shall not project more than 3 inches beyond the pitch line of joints.

4. No course shall be less than 12 inches thick, nor overlie one of less thickness.

5. The stones shall be rectangular in form.

Stretchers.

6. No stretcher shall be less than 3 feet long, and in no case shall the upper or lower bed be less than one and one-half times the thickness of the course.

Headers.

7. Headers shall not be less than 4 feet long, nor less than three times the thickness of the course, and shall be at least as wide as high. Not less than 30 per cent. of the face of the wall shall consist of headers. Headers and backing shall be so arranged as to form a bond entirely through the pier.

8. All stones shall be laid with close joints on their quarry beds, the largest bed down, and the beds and tops dressed to parallel faces throughout their entire extent.

Joints. 9. The joints shall be vertical and horizontal, and the vertical joints shall break not less than 12 inches. Vertical joints shall be dressed close for at least 12 inches back from the face.

10. No horizontal or vertical joints shall be more than ½ of an inch in thickness.

Backing. 11. The backing shall be of such shape and dimensions as will closely fill the interior of the pier and thoroughly bond the outside courses with each other. No spalls will be allowed except in small openings in the backing.

12. The face-stones of each course shall be laid in full mortar beds, and brought to a bearing with a wooden maul. The interior of the course shall be thoroughly grouted and leveled off before the next course is laid.

Dowels. 13. The face-stones of the rounded ends of piers shall be drilled through before setting, and the drill holes extended six inches into the course below. One-inch round iron dowels, 12 inches long, shall be driven into these holes with a small quantity of mortar.

Coping. 14. The coping shall be dressed throughout, and at least half the stones shall extend across the pier. The face of each stone shall have a margin draught on upper and lower edges. The coping and all pedestal stones and draw-rests shall be bush-hammered where required by the engineer. Chisel-draughts, 1½ inches wide, shall be made at all angles in the masonry.

VII. Slope Wall. Slope wall shall consist of flat stones laid close, for protection of banks, of same general size as specified for rip-rap, and built of such thickness and slope and in such manner as the engineer may direct.

VIII. Mortar. 1. All mortar or grout shall be composed of one part of good hydraulic cement and two parts of coarse, clean, sharp sand, thoroughly mixed, and tempered as used.

Cement. 2. The cement shall be hydraulic, of uniform quality, and equal in all respects to the best Louisville or Rosendale cement. It shall be newly made, fine ground, and capable of standing a tensile strain of 40 pounds per square inch of section when mixed pure and made into test bars, exposed thirty minutes in air and twenty-four hours under water.

3. Samples for testing shall be furnished when required by the engineer. All packages or barrels containing the cement shall have the maker's brand carefully marked thereon.

4. All cement shall be kept dry until used.

IX. Quarries and Transportation. 1. The quarries will be such as are approved by the engineer, along the line of the St. Louis and San Francisco Railway, between Seligman, Missouri, and Van Buren, Arkansas; or on the line of the Eureka Springs Railway, near Beaver Station, or on the line of the Little Rock and Fort Smith Railway.

2. The railway company will furnish free transportation for stone from points named on its own line of road or on the Eureka Springs

Railway to Van Buren, but the contractor shall pay for transportation over the Little Rock and Fort Smith Railway, or from the quarries to the nearest side track by teams or otherwise.

3. Stone, however, for rip-rap, concrete and slope-wall, shall be taken from quarries near the bridge site and transported by the contractor at his own cost and expense.

4. The contractors shall furnish all tools, machinery and plant of every description—both at the quarry and at the bridge site.

5. The stone shall be cut at the quarries, and the contractor shall promptly load and unload the cars furnished for transportation of stone or other materials.

6. The railway company will furnish free transportation for timber from stations south of West Fork, Arkansas, to Van Buren, over its own line of road.

7. The railway company will also furnish free transportation between stations on its own line of road for men, tools and material and supplies actually needed for this work, except as more particularly specified for stone and timber.

X. Temporary Works.

1. No material shall be measured or included in the estimate which does not form a part of the permanent structure.

2. And all such material or temporary works shall be removed as directed by the engineer, leaving the bridge free and unobstructed on completion of the work.

XI. Defective Material.

All material furnished shall be sound, and free from any defect that will injure its strength or durability, and subject to the inspection and acceptance of the engineer before being used. If rejected by him it shall be promptly removed.

XII. General.

1. The contractor shall not sub-let or transfer his contract, or any part thereof, without the written consent of the engineer.

2. The contractor shall, at his own cost and expense, take down, remove and rebuild any work which, in the opinion of the engineer, has been imperfectly executed.

3. All extra work other than that to be paid for as herein set forth required by the engineer, and not embraced in the items and prices set forth in the contract, shall be done by the contractor only on the written order of the engineer, who shall return with each monthly estimate a bill for the same, based on the actual cost of labor and material used, such as in his judgment is reasonable and just. And 10 per cent. shall be added to the same for the use of tools, superintendence and all other expenses. The acceptance of such bill by the contractor shall be deemed and taken as waiving any further claims for or on account of extra work done during the month referred to in said bill.

4. No charge shall be made by the contractor for hindrances and delays from any cause whatever in the progress of the work, but it may entitle him to an extension of the time allowed for the completion of the work

equal in length to the detention, to be determined by the Chief Engineer of the St. Louis and San Francisco Railway Company; provided, he shall give the said chief engineer immediate notice in writing of the cause of the detention.

XIII. Form of Bid. 1. Bids will be received for the substructure hereinbefore as specified either with a lump sum to be paid for the whole such substructure complete; or

2. With specific prices to be paid for the several classes of work as enumerated below.

3. Said prices shall cover and include all material, barges and machinery or other plant, tools, labor, temporary work or other expense used or incurred in the progress of this work.

4. No constructive or conventional measurement will be allowed.

5. Quantities of timber in estimates shall include only the actual cubic contents of the timber turned into board measure, and as placed permanently in the structure.

6. Quantities of masonry shall include only the actual cubical contents of the masonry inside the draft lines and according to the plans.

XIV. The contractor shall employ persons experienced and skilled in the various kinds of work specified; and shall use the best tools and machinery to be obtained for rapid and efficient progress.

XV. Any or all of the work herein specified shall be carried on night and day if so directed by the engineer. In which case the contractor shall, within ten days after being so directed, provide at his own cost and expense sufficient electric or other artificial light, and a force adequate to keep three full shifts of men successively employed during the whole of each twenty-four hours; and shall maintain such force until, in the judgment of the engineer, the necessity for so doing shall have passed.

XVI. Engineer. Whenever the word engineer occurs in these specifications, it shall have reference to the Chief Engineer of the St. Louis and San Francisco Railway or his authorized assistants on the work.

XVII. 1. Timber in caissons shall include all timber of every kind furnished, framed, and placed in caissons which are sunk to bed-rock.

2. Timber in cribs and open caissons shall include timber of every kind (excepting piles and grillage) furnished, framed and placed in cribs and open caissons for foundations of masonry.

3. Timber in grillage shall include all caps for piles and timber, or platforms resting thereon, or other platforms for foundations of masonry.

4. Piling shall include only such part of the piles as remain in the foundations of the masonry and below the level of the bottom of the grillage resting on them, and such price includes furnishing, driving and cutting off the tops of the piles level as hereinbefore specified.

5. Sinking caissons shall be measured per foot, vertically from the average level of the bottom of the cutting edge to low-water level as

already established by the engineer, and shall include all excavation from the interior of the caissons or elsewhere of all classes of material encountered in sinking them excepting the bed-rock.

6. Wrought-iron shall include all wrought-iron bolts and their attachments, drift-bolts, dowels, washers, plates, cutting-edges, nails, spike or other wrought-iron furnished and used in the foundations and masonry and remaining in the permanent structure.

7. Cast-iron shall include all cast washers, plates or other castings furnished and used in the permanent structure.

8. Concrete in working chamber of caisson shall include only such concrete as is furnished and used in the working-chamber proper of the caissons sunk to bed-rock.

9. Excavations above water shall include all excavations of foundations and other excavations incidental thereto, and will include all classes of material except as hereinafter more particularly specified.

10. Excavations under water shall only apply to excavation of foundation pits and deepening of channels under water where required by the engineer, and shall cover all classes of material encountered, and include draining, bailing, pumping, and all material, labor or other expense connected with such excavation.

XVIII.

1. The river shall, at all times during the construction and erection of the substructure, be kept free for navigation.

2. All coffer-dams, staging and other obstructions must be removed by the contractor when directed by the engineer, leaving the river entirely unobstructed except by actual space occupied by the masonry.

XIX. Bed rock.

1. In case the elevation of the underlying bed-rock above datum line of levels is less than that shown on plans, the contractor shall be entitled to the sum of $400 per vertical foot for the difference of level so found at the sites of Piers 1, 2, 3, 5 and 6, and the sum of $600 per vertical foot at the site of Piers No. 4.

2. In case the elevation of the underlying bed-rock above datum line of levels is greater than that shown on plans, the railway company shall be entitled to a credit of $400 per vertical foot for the difference of level so found at the sites of Piers 1, 2, 3, 5 and 6, and the sum of $600 per vertical foot at the site of Pier No. 4.

ST. LOUIS AND SAN FRANCISCO RAILWAY.—VAN BUREN BRIDGE.—FINAL ESTIMATE.

| | Timber in Caissons. | Timber in Cribs. | Timber in Grillage. | Piles in Foundation. | Sinking Caissons. | Wrought-iron Foundation and Masonry. | Cast-iron in Foundations. | Concrete in Air Chamber. | Concrete in other Foundations. | Bridge Masonry. | Rip-rap. | Excavation Above Water. | Excavation Under Water. | Bridge Iron. | Bridge Steel. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Ft. B.M. | Ft. B.M. | Ft. B.M. | Lin. ft. | Ver. ft. | Pounds. | Pounds. | Cu Yds. | Cu Yds. | Cu.Yds. | Cu.Yds. | Cu.Yds. | Cu.Yds. | Pounds. | Pounds. |
| Pier No. 1 | 28 756 | 7 956 | .... | .... | 11.55 | 3 903 | 225 | 68.3 | 78.2 | 379.8 | .... | 1 933 | | | |
| 2 | 37 400 | 18 168 | .... | .... | 24.23 | 6 066 | 300 | 103. | 158.2 | 436.2 | | | | | |
| 3 | 37 400 | 39 629 | .... | .... | 32.47 | 8 270 | 300 | 91. | 337.1 | 491.0 | | | | | |
| 4 | 81 323 | 58 502 | .... | .... | 32.84 | 9 668 | 300 | 198.4 | 721.6 | 987.4 | | | | | |
| 5 | 37 400 | 43 906 | .... | .... | 31.46 | 8 496 | 300 | 91. | 377.5 | 471.0 | | | | | |
| 6 | 37 400 | 41 559 | .... | .... | 31.80 | 8 202 | 300 | 91. | 358.4 | 434.2 | | | | | |
| 7 | .... | 29 764 | 11 233 | 2 528 | .... | 3 221 | .... | .... | 89.2 | 385.7 | 431 | .... | 225 | | |
| 8 | .... | 29 764 | 11 233 | 2 319 | .... | 3 378 | .... | .... | 93.7 | 386.7 | 221 | 80 | 410 | | |
| 9 | .... | 29 764 | 11 233 | 2 210 | .... | 3 171 | .... | .... | 136.0 | 378.5 | 270 | 200 | 480 | | |
| 10 | .... | .... | .... | .... | .... | ..... | .... | .... | 68.8 | 173.4 | .... | 428 | | | |
| Span No. 1 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 237 452 | 196 009 |
| 2 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 237 452 | 196 009 |
| 3 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 540 483 | 220 576 |
| 4 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 237 452 | 196 009 |
| 5 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 129 102 | 86 147 |
| 6 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 129 102 | 86 147 |
| 7 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 129 102 | 86 147 |
| 8 | .... | .... | .... | .... | .... | ..... | .... | .... | .... | .... | .... | .... | .... | 129 102 | 86 147 |
| Total | 259 681 | 299 012 | 33 699 | 7 057 | 164.35 | 54 375 | 1 725 | 642.7 | 2418.7 | 4523.9 | 922 | 2 641 | 1 115 | 1 749 247 | 1 153 191 |

# DISCUSSION.

CHARLES MACDONALD, M. Am. Soc. C. E.—The interest attaching to bridges of the character described in this paper, is perhaps more of a commercial than professional nature. There were really no engineering difficulties worth mentioning, in the construction. The rock was about 30 feet below the water, overlaid with sand, and easily reached by pneumatic caissons in that portion of the river where scour was liable to take place. On the south side pile foundations were used, for the reason that a large excess of waterway had been provided for, and there was no probability of scour at those points. I may mention a point, connected with the testing of cement, which might be of interest to the Committee on Tests of Cement. The cement was made at Louisville, Ky. The first car load received was carefully tested, and about 10 per cent. of the barrels were reported as falling below the specifications. For the purpose of verifying these tests, I had two samples taken from each of the rejected barrels. The first set was mixed in exact accordance with the practice which had been observed in the original tests; and was tamped in a briquette, with a moderate amount of pressure.

Tests made upon these samples all fell below the specifications. The second set of samples was treated in the same way, except that the tamping was a little more decided than in the first case. Every one of these tests came above the specifications. It appeared evident that the pressure exerted in tamping was the controlling element, and that a slight difference in amount would give very different results on the test piece. It would seem as if the most important question, in connection with testing cement, is the manner of preparing the test pieces.

In sinking caisson No. 3 an interesting experience was encountered. After the caisson had been sunk a few feet into the sand, the water in the river rose rapidly, completely covering the work, and obliging a suspension of operations.

During the high water the sand scoured out under the up-stream end of the caisson, until it dropped down 6 feet out of level and the whole caisson moved 2 or 3 feet down stream.

After the water had receded sufficiently to renew operations, Messrs. Sooysmith, who had undertaken to do the pneumatic work, succeeded in righting the caisson, and, at the same time, crowding it back into place by a system of inclined structures placed in the working chamber on the up-stream side of the center. Upon releasing the air pressure suddenly the caisson descended and worked up stream at the same time.

Mr. Sooysmith will doubtless be able to describe this operation more in detail.

It was an ingenious method of overcoming what appeared to be a serious difficulty.

CHARLES SOOYSMITH, M. Am. Soc. C. E.—When we are working in a location where the ground will permit us to pile the material higher on one side than on the other, the most effective way to move the caisson is to bring about a greater pressure on one side than on the other, and this is done by simply piling the material higher on that side. When we had about 40 or 50 feet of material about the caisson, we found that a difference of 5 or 6 feet in the height of the material on one side had a tendency to work the caisson away from the high side. Throughout the sinking of the caissons we often change the height of the material every day or so and find it very effective.

We had an interesting case at the Randolph Bridge over the Missouri, at Kansas City. A channel caisson had gotten 2 or 3 feet out of position when it was but a few feet from the rock; the masonry had been started on it and it was very desirable to get it more nearly in position. There was too much material around it to enable us to push it, and there was not enough distance still to sink to permit us to move it in the manner just described. It was moved into position in the following way: 12 x 12 timbers resting on plank platforms on the sand were set up inside the caisson; these were all inclined in the direction in which we wished to move the caisson. After excavating under the cutting edges a portion of the pressure was let off, bringing the weight of the caisson on the inclined posts, and the result was that the caisson moved slightly in the direction we wished it to go, and two or three repetitions of this method brought it into position.

F. COLLINGWOOD, M. Am. Soc. C. E.—In sinking the Brooklyn caisson of the East River Bridge, there was a decided excess of pressure on the land side. The ultimate depth reached was 44½ feet below mean high tide, and the surface of the earth on the land side was about 6 feet above the water surface. The bottom had been dredged to 18 feet depth, and sloped to a greater depth at a short distance in the stream. The resultant of the weight of the caisson and this difference of side pressure was, therefore, outward, and when the full depth was reached the caisson was found to have moved two feet towards the river.

The caisson had been made purposely abundantly large (for this, among other reasons), and as the masonry base was also made large, there was no difficulty in adjusting the tower above water to its true position.

As the resultant from the cables was also outward, the final effect was favorable rather than otherwise.

JOHN BOGART, M. Am. Soc. C. E.—Did the piers of the Poughkeepsie Bridge get out of line at all?

T. C. CLARKE, M. Am. Soc. C. E.—The caissons of the Poughkeepsie Bridge were sunk to a depth of 136 feet. We found that the river-bed consisted of soft sand, so soft that everything that sunk disappeared at

once. The soil grew harder as it went down until some gravel was found with it, and just before reaching the bed-rock it turned into a very hard, solid gravel.

The piers of the Poughkeepsie Bridge were very large; the base was 60 feet wide by 100 feet long and they were about 130 feet high. We made the size on the bottom 60 by 100, which gave a pressure on the bed of the river of about 3¾ tons to the square foot. Then, for the reason that so much size was not required as we went up, we drew this in to 100 by 50 feet on the top. Whether this had anything to do toward keeping the bridge piers perpendicular I think no one could tell. I am certain of one thing, that our experience there shows that we must not overload the piers if we want them to go down without deviating from their normal position. We must keep the excavation below the cutting edge. When the weight overcame the side friction the thing dropped just like a plumb-bob. But, as a matter of fact, they did not get out of line.

JOSEPH P. DAVIS, M. Am. Soc. C. E.—Can you give any data as to the friction of the sand on the side of the caisson?

Mr. CLARKE.—I think, if my memory is correct, it took, to overcome the friction, a weight of about 800 pounds per square foot, between 700 and 800 pounds to the square foot. I am not quite certain. On referring to my note books I find that when the caisson was sunk 45 feet below the river-bed its perimeter was 320 feet. $45 \times 320 = 14\,400$ square feet of exterior rubbed surface. Its immersed weight was then 8 000 tons = 16 000 000 pounds. $\frac{16\,000\,000}{14\,400} = 1\,100$ pounds per square foot of rubbed surface, equals the downward pressure. This was not enough to make the caisson move, until a hole was dredged below it. When that hole was dredged, and the support under the base taken away, the caisson sank down. Hence the friction on the rubbed surface could not have been as much as 1 100 pounds. We called it 700 or 800 pounds, but it is not possible to state it exactly.

Mr. COLLINGWOOD.—Was there any increase as you went down?

Mr. CLARKE.—No; we could not see that there was.

C. D. PURDON, M. Am. Soc. C. E.—I may mention, in connection with the subject of caissons being out of place, that at the Rob Roy Bridge (over the Arkansas River on the St. Louis, Arkansas and Texas Railway), where cylinders of 7 feet diameter were sunk to a depth of 60 feet below low water, the method employed to bring a cylinder back to place was to first get it out of plumb, with the bottom pointing in the direction in which it was necessary to move it; to sink it thus inclined until calculations of inclination and position together showed that the bottom was in the right place, and then to plumb it; this was done with twelve cylinders.

To throw the cylinders out of plumb and afterwards to plumb them, pieces of 12 × 12 inch timber were clamped on the cylinders, and inclined struts placed from these to either sills on the sand-bar or to caps on clumps of piles driven for the purpose.

At the Junction Railway Bridge, over the Arkansas River, at Little Rock, Ark., of which I also had charge, one of the caissons was brought back to its place by sinking it out of plumb; it was thrown out of plumb and righted by simply excavating below the cutting edge on one side only.

This caisson was but little out of place, however.

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The following are the illustrations of the paper by Mr. Purdon on the Van Buren Bridge:

| | | |
|---|---|---|
| Plate | XVIII. | Photograph of Bridge. |
| " | XIX. | Location. |
| " | XX. | Triangulation. |
| " | XXI. | Caisson and Crib of Fixed Piers. |
| " | XXII. | " " " |
| " | XXIII. | Pier 2. |
| " | XXIV. | Water Gauge Profile. |
| " | XXV. | Ravine Section. |
| " | XXVI. | Process of Leveling Caisson of Pier 3. |
| " | XXVII. | Caisson and Crib of Pier 4. |
| " | XXVIII. | Pier 4; Elevation and Section. |
| " | XXIX. | Plan of Courses, Pier 4. |
| " | XXX. | " " " |
| " | XXXI. | " " " |
| " | XXXII. | " " " |
| " | XXXIII. | Pier 7. |
| " | XXXIV. | Pier 10. |
| " | XXXV. | Profile showing Sinking of Caisson. |
| " | XXXVI. | Profile showing Progress of Masonry. |
| " | XXXVII. | Superstructure of 252-feet 9-inch Spans. |
| " | XXXVIII. | Superstructure of 162-feet Spans. |
| " | XXXIX. | Superstructure of Draw Span. |

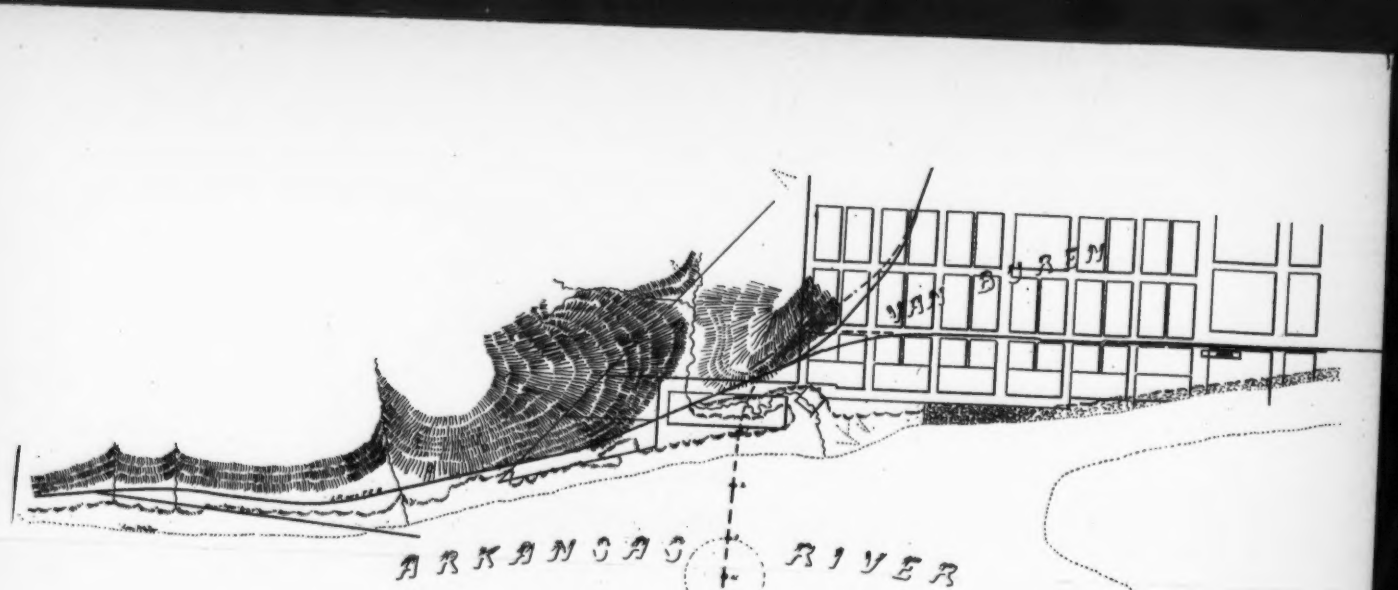
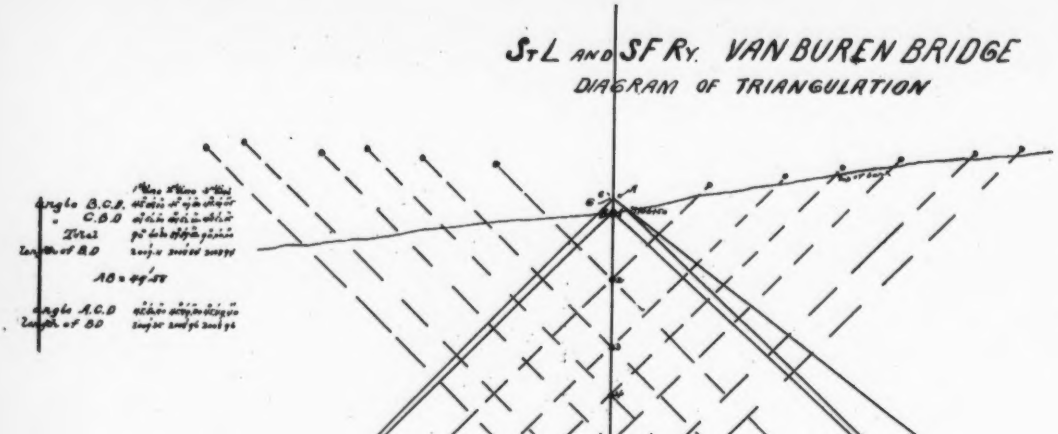

ARKANSAS RIVER

PLATE XIX
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412
PURDON ON
VAN BUREN BRIDGE.

ARKANSAS RIVER

# St L and SF Ry. VAN BUREN BRIDGE

## DIAGRAM OF TRIANGULATION



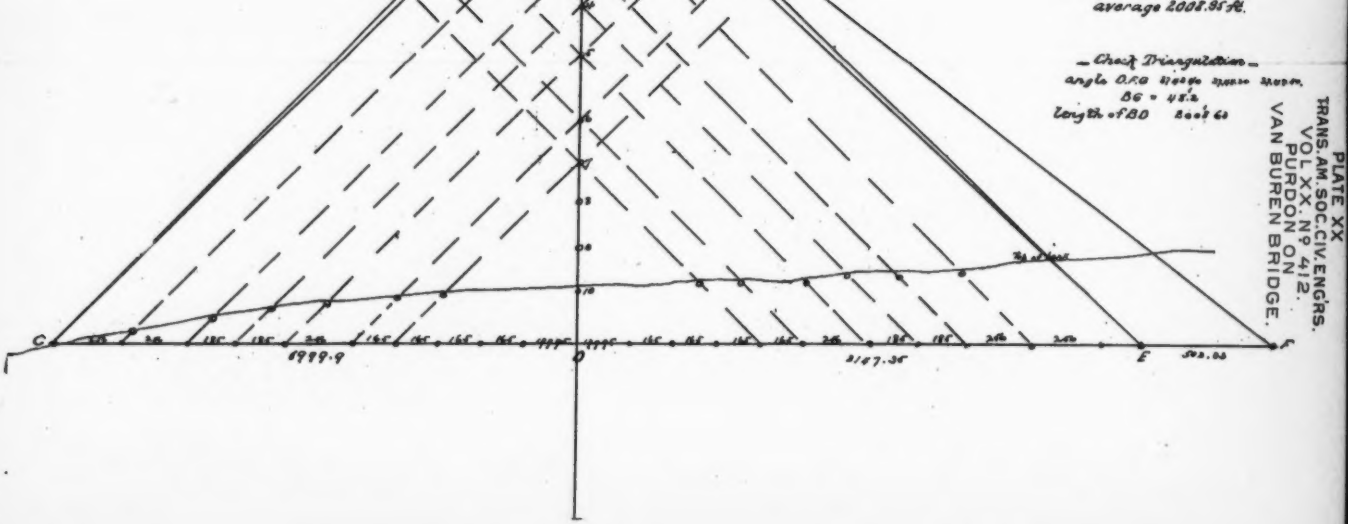



PLATE XX
TRANS. AM. SOC. CIV. ENGRS.
VOL. XX. NO. 412.
PURDON ON
VAN BUREN BRIDGE.

# CAISSON AND CRIB OR FIXED PIERS

Size of caisson 18' by 39' over all
Timbers 12"×12" and 3"×12"
Height of crib varies to suit level of rock

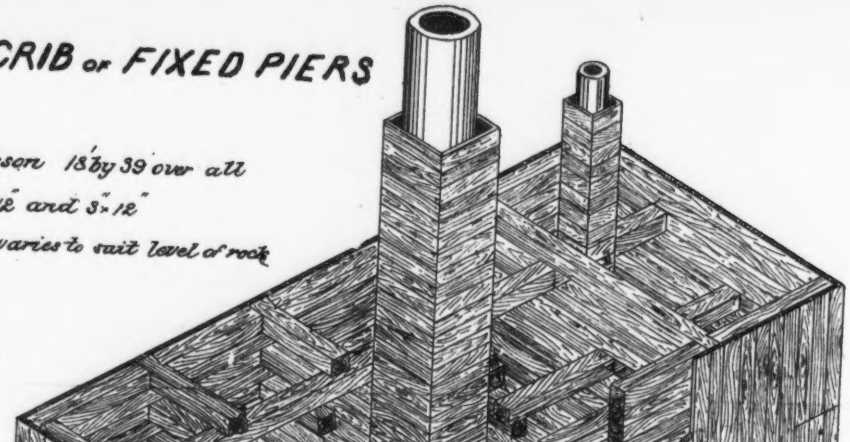
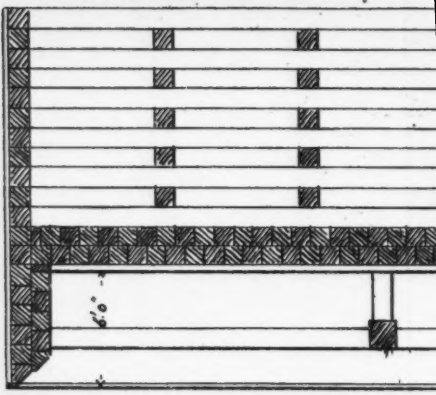
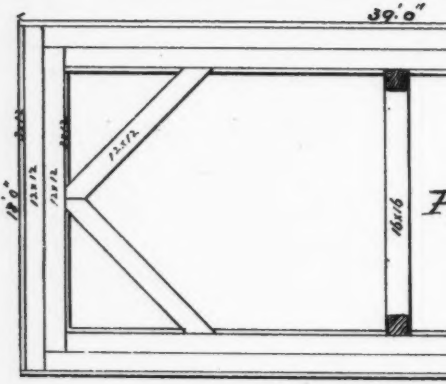


PLATE XXI
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.

# CAISSON AND CRIB OF FIXED PIERS



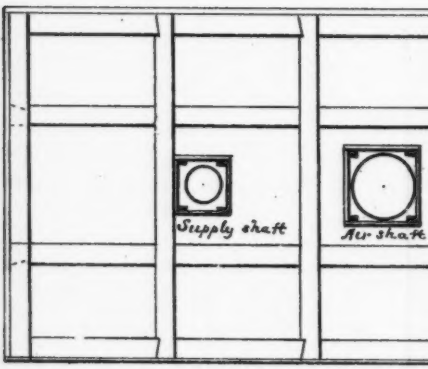
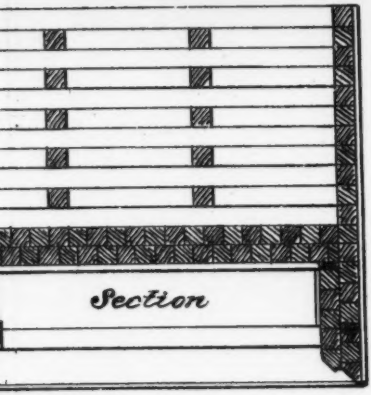
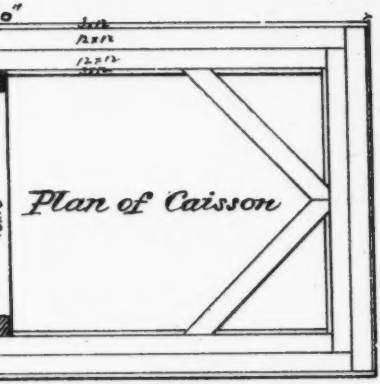
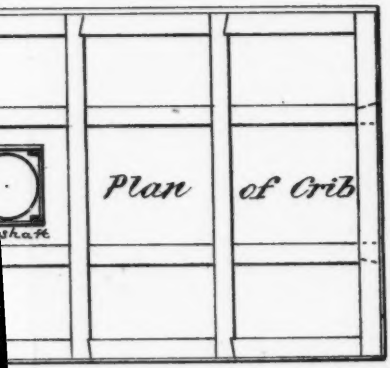



PLATE XXII
TRANS. AM. SOC. CIV. ENGRS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.



PIER 2

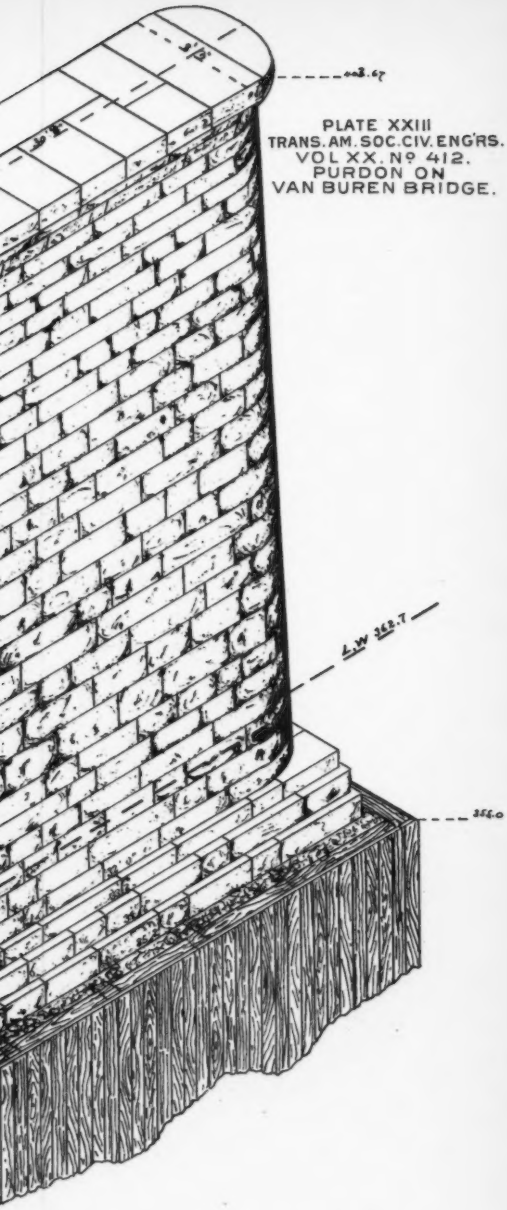
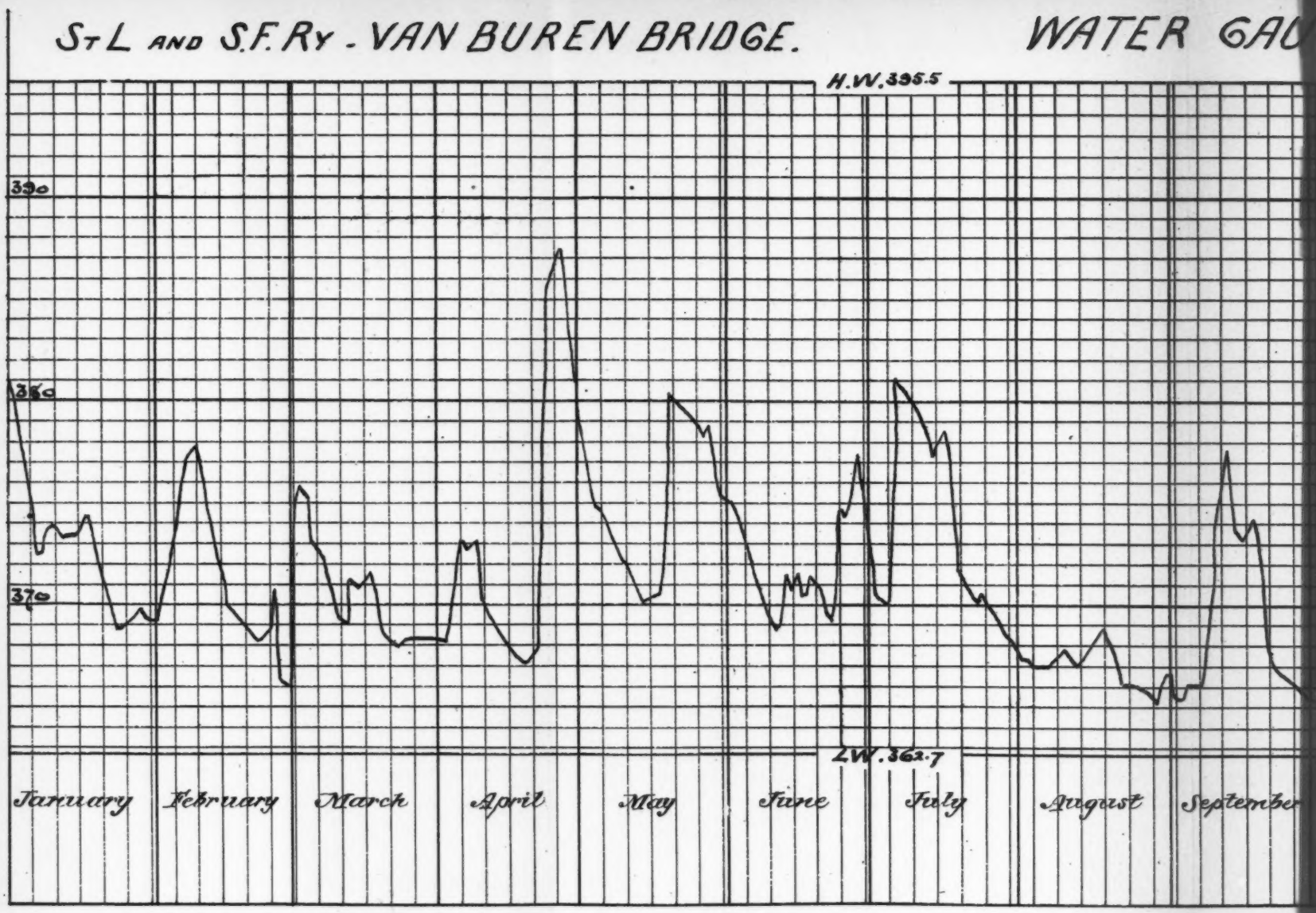


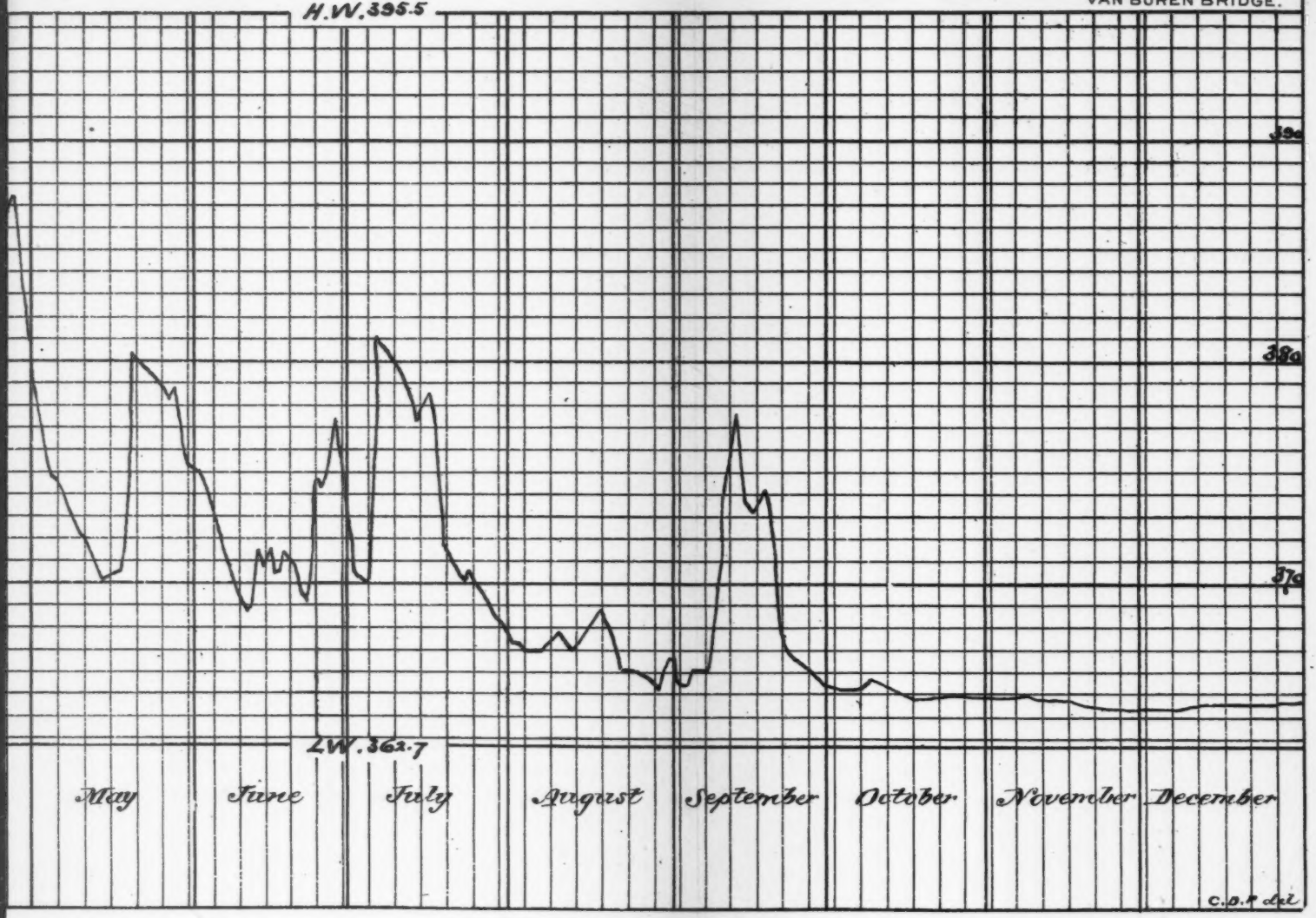
PLATE XXIII
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.
L.W 362.7
356.0


St L and S.F. Ry - VAN BUREN BRIDGE.
WATER GAU
H.W. 395.5
390
380
370
L.W. 362.7
January
February
March
April
May
June
July
August
September

# EN BRIDGE.

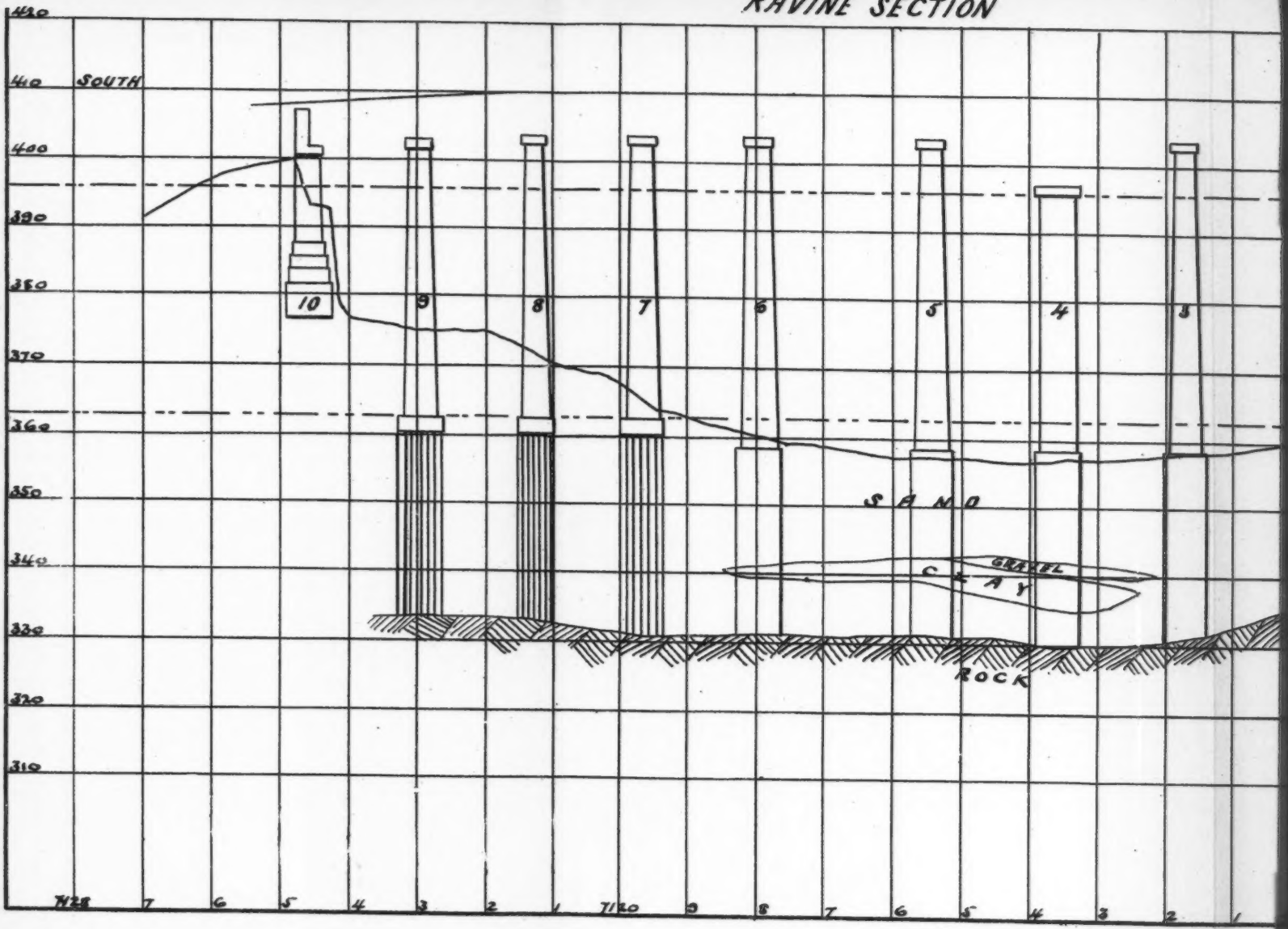
# WATER GAUGE 1885

PLATE XXIV
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.



# St L and S.F. Ry - VAN BUREN BRIDGE

## RAVINE SECTION



# St L and S.F. Ry. - Van Buren Bridge

## Ravine Section

PLATE XXV
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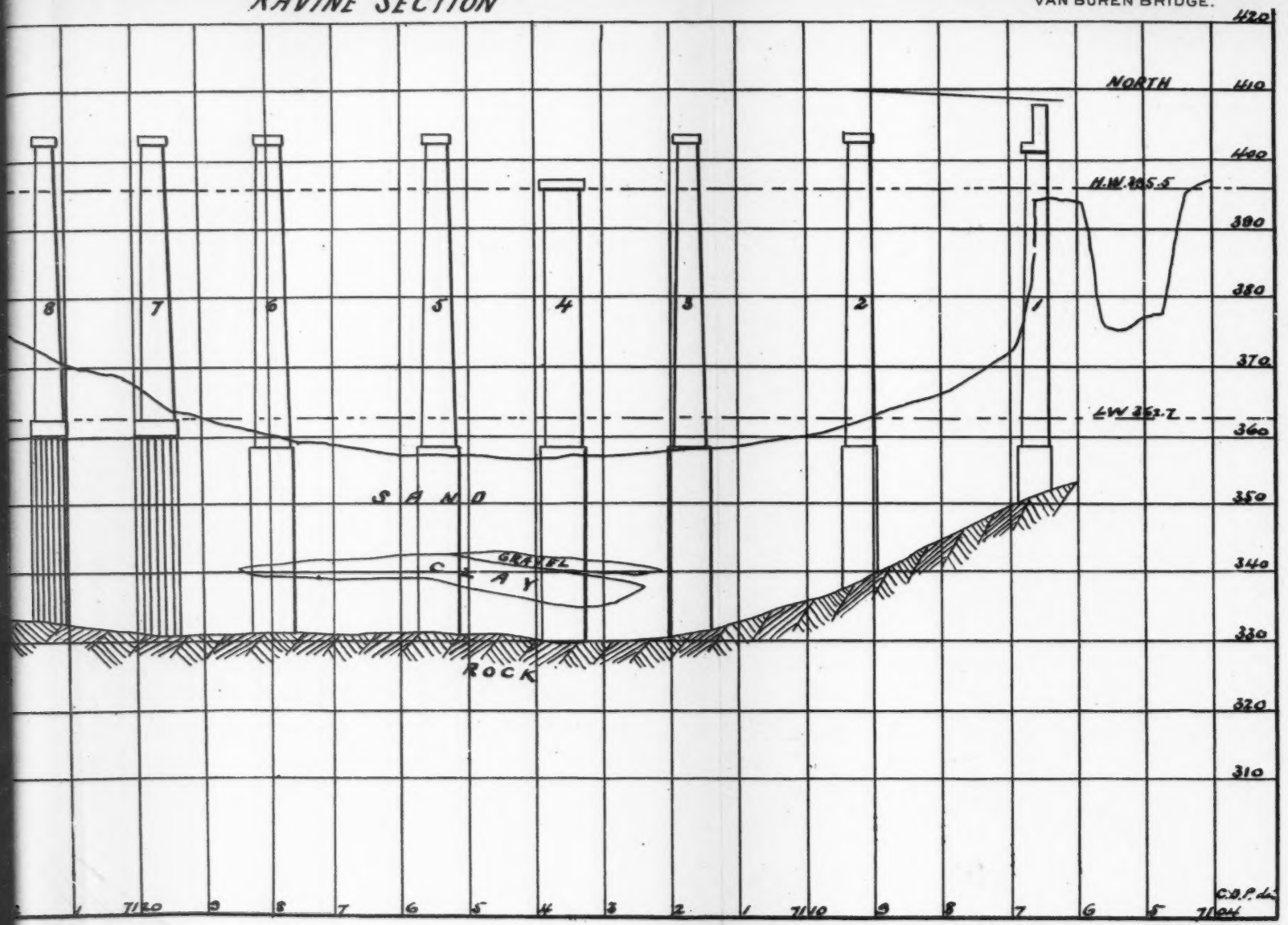
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.



*Process of levelling Caisson of Pier 3*

*8ft - 1in*

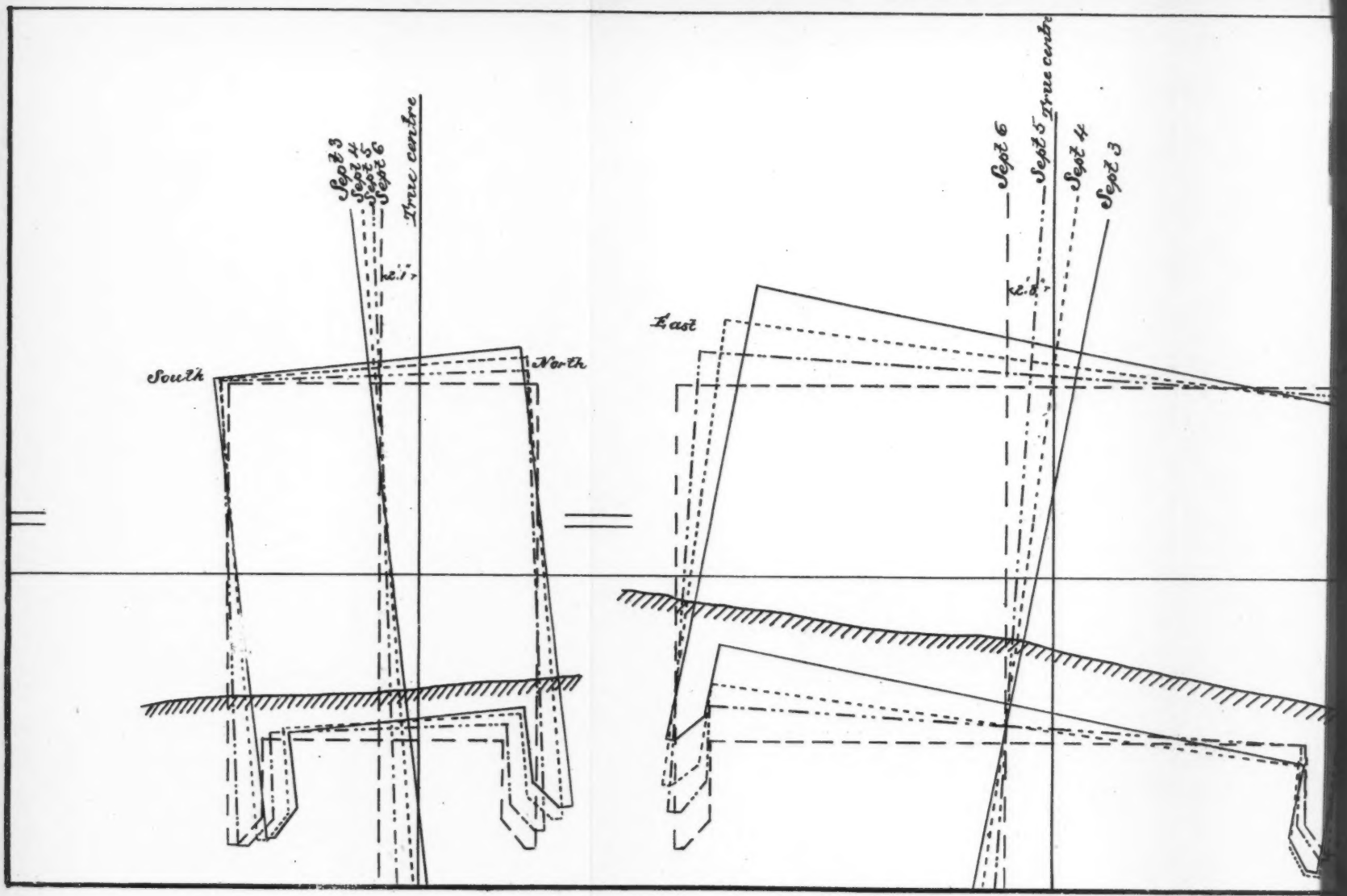
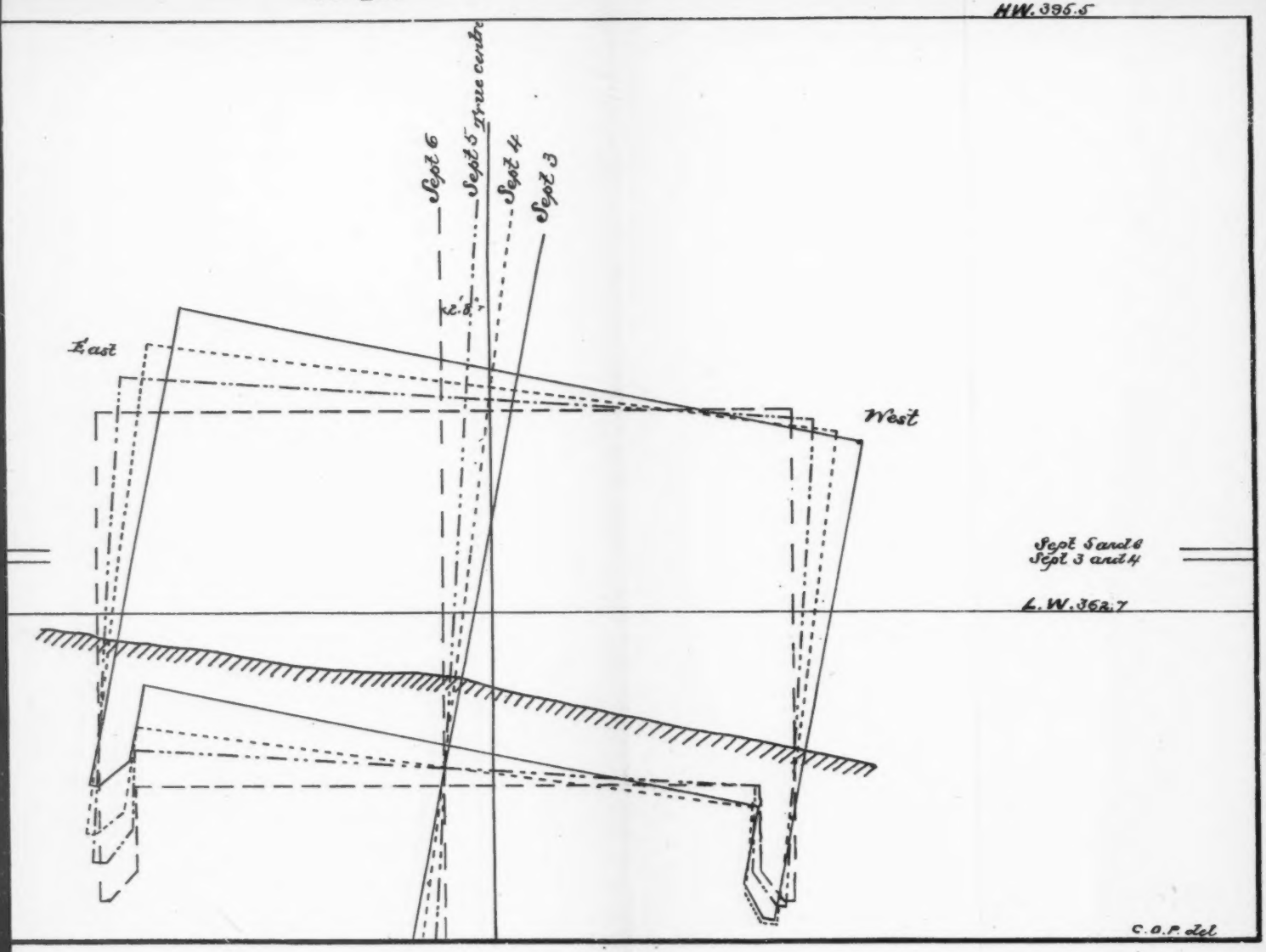


PLATE XXVI
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.

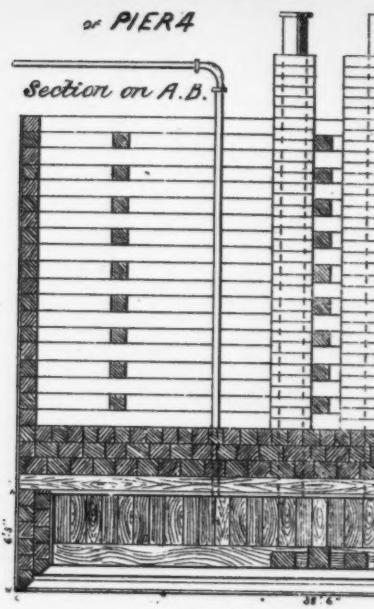
# Process of levelling Caisson of Pier 3

8ft - 1in



CAISSON AND CRIB.

OF PIER 4



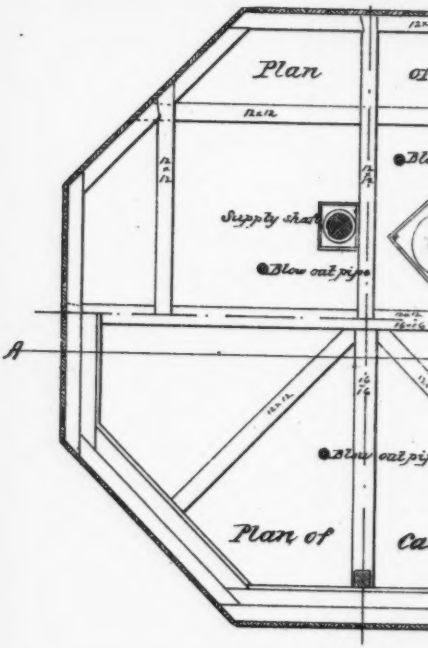
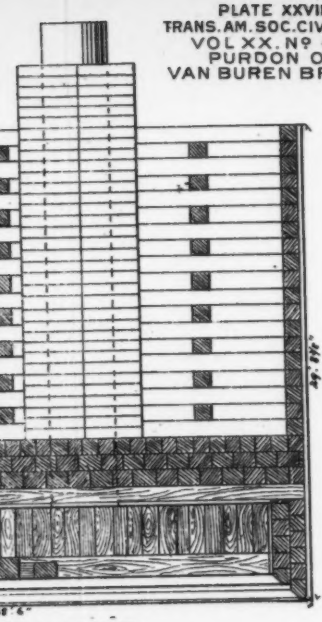
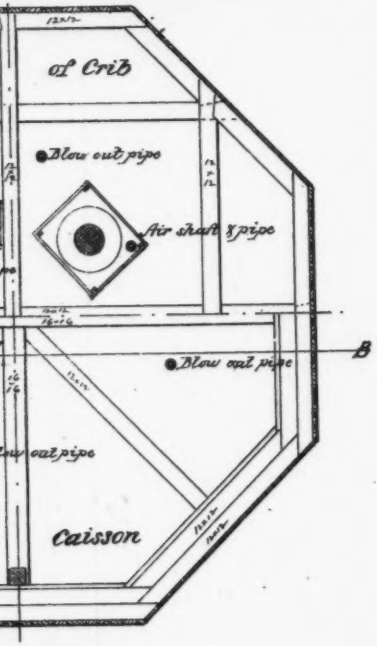



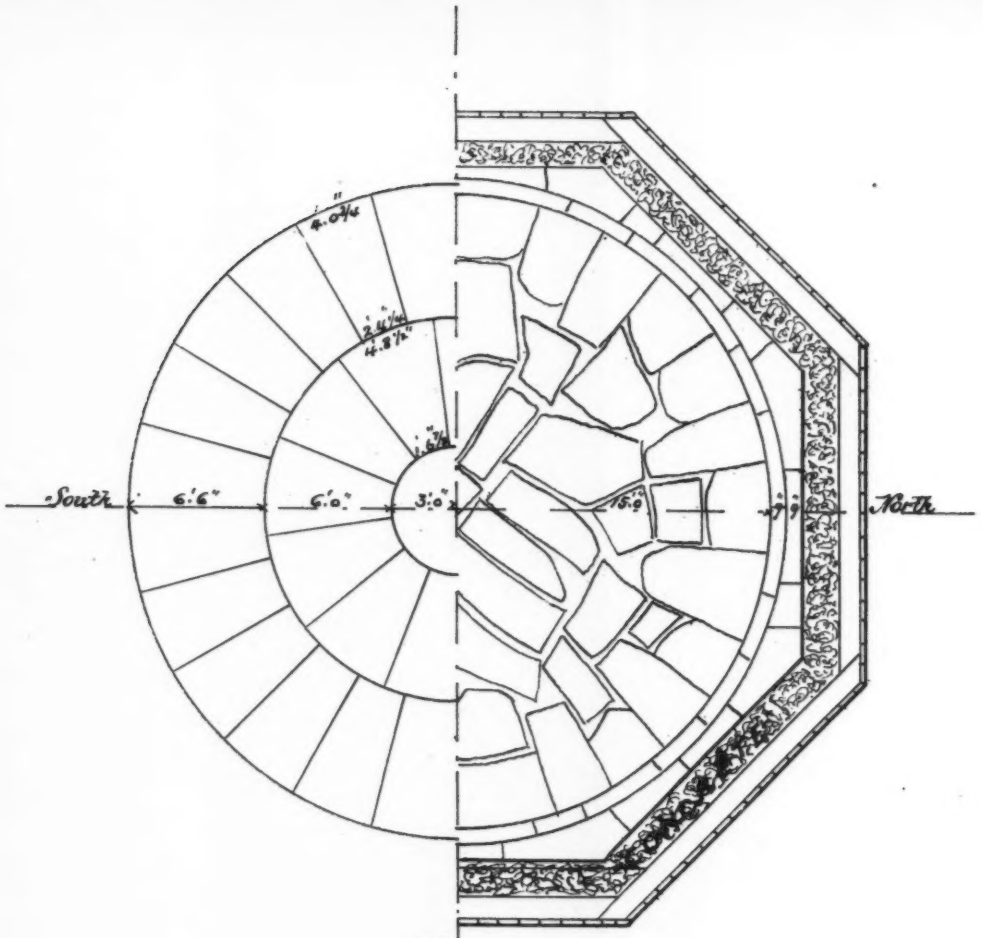
PLATE XXVII
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.





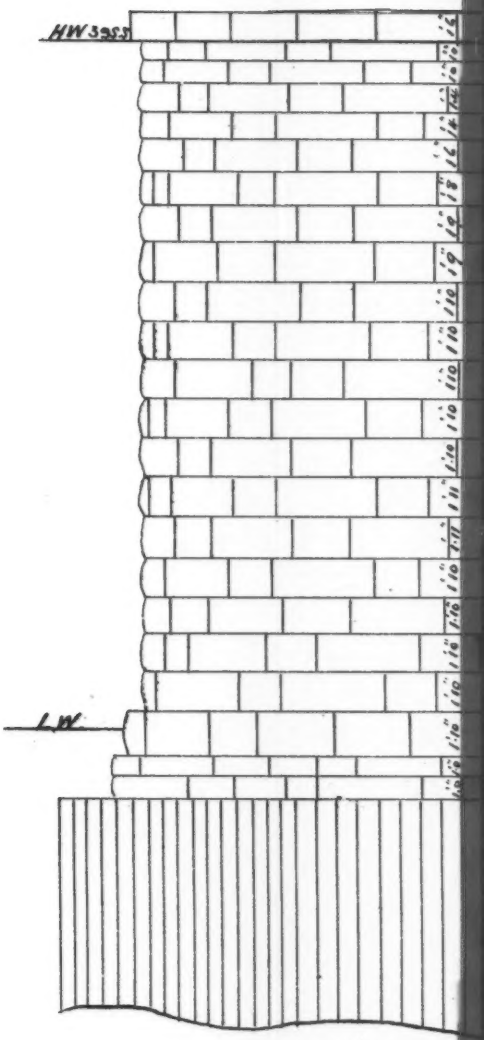
# PIER 4 AS BUILT

8A = 1in




Plan

Coping . Footings

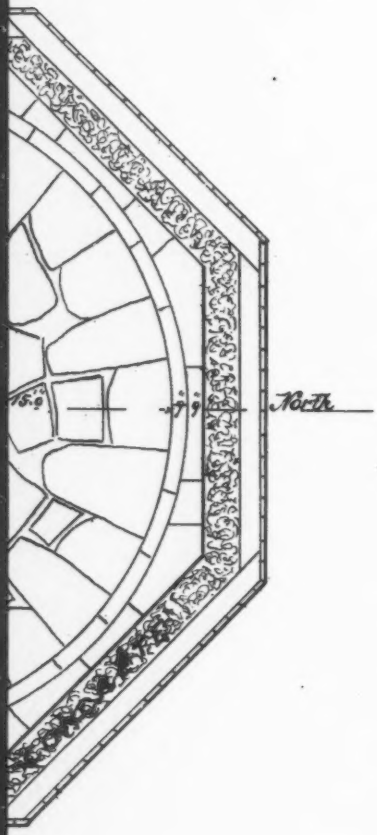


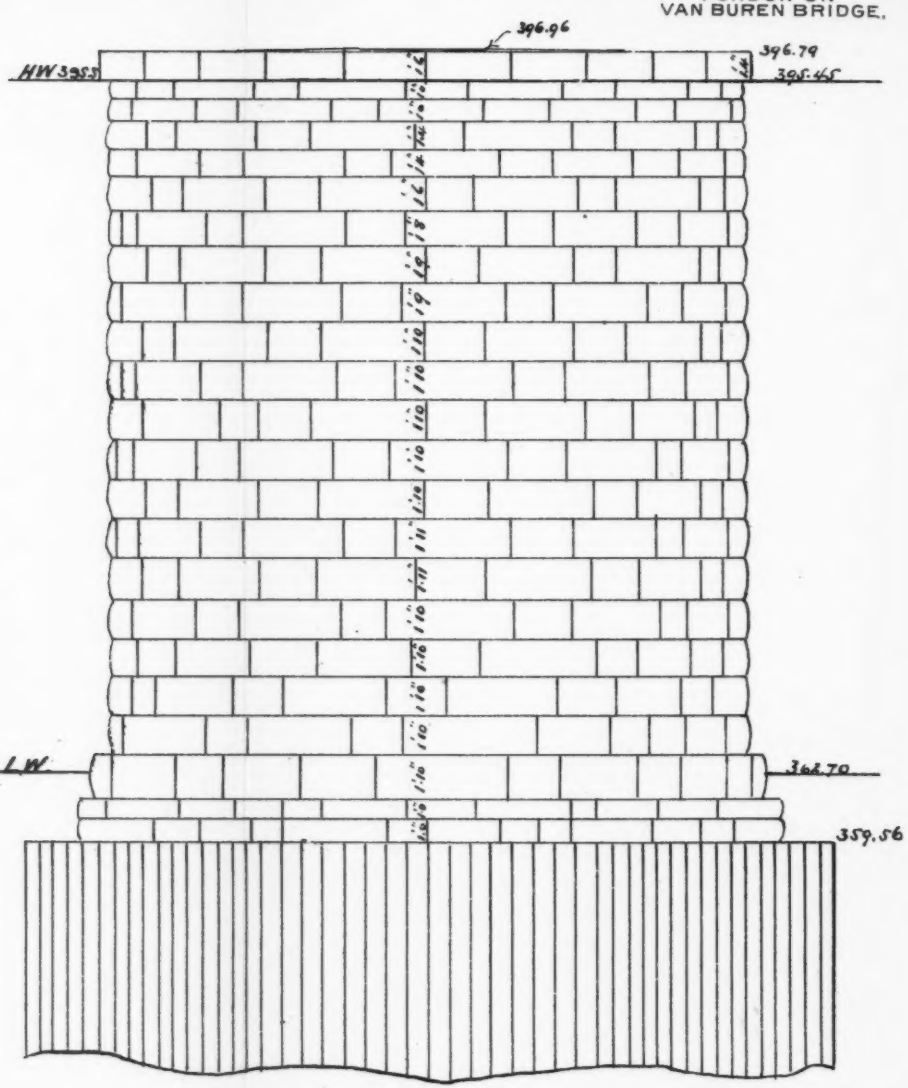

East El

# PIER 4 AS BUILT

8 ft = 1 in

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. N° 412.
PURDON ON
VAN BUREN BRIDGE.

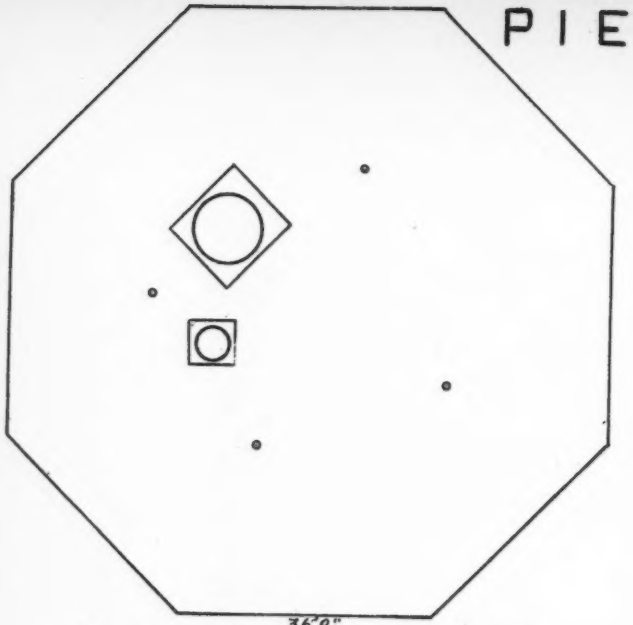



East Elevation

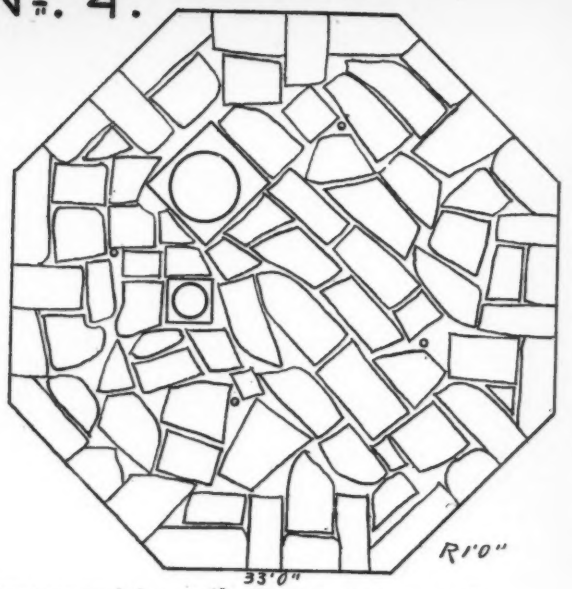
# PIER No. 4.

Concrete



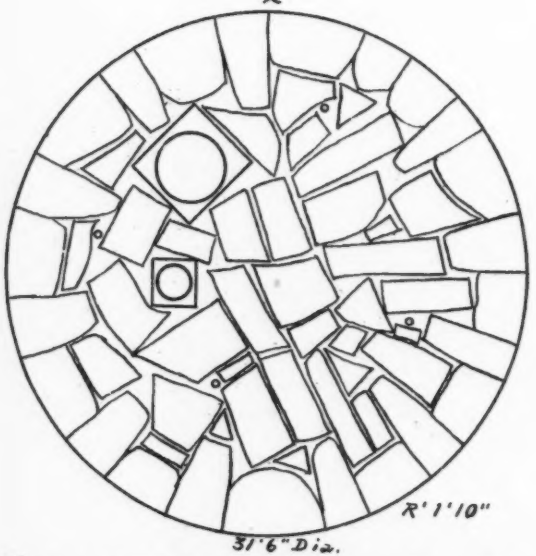

Commenced July 30th Finished Aug 10th

Course # 1- A



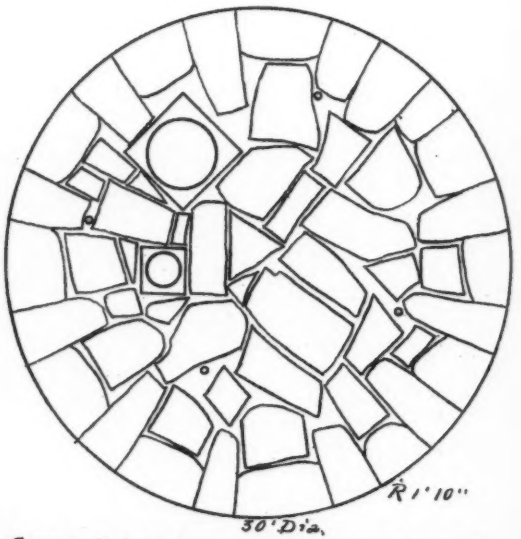

Commenced Aug 11th Finished Aug. 13th
Cement 27 1/2 bbls.

2




Commenced Aug. 14th Finished Aug. 15th
Cement 33 bbls.

3

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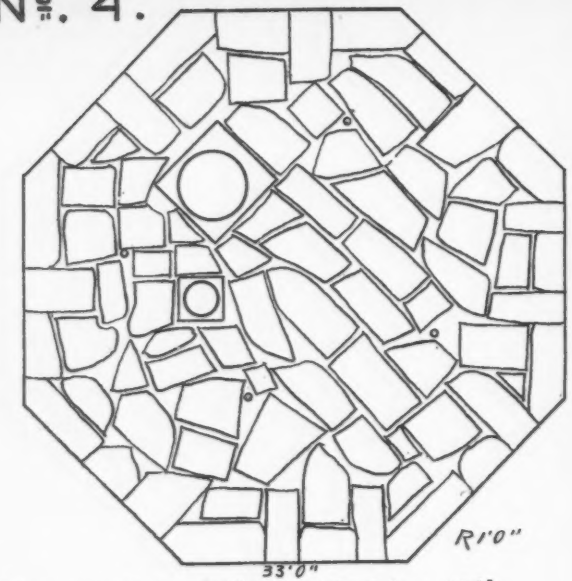
Cement 31 bbls.

R Nº. 4.

Course # 1- A

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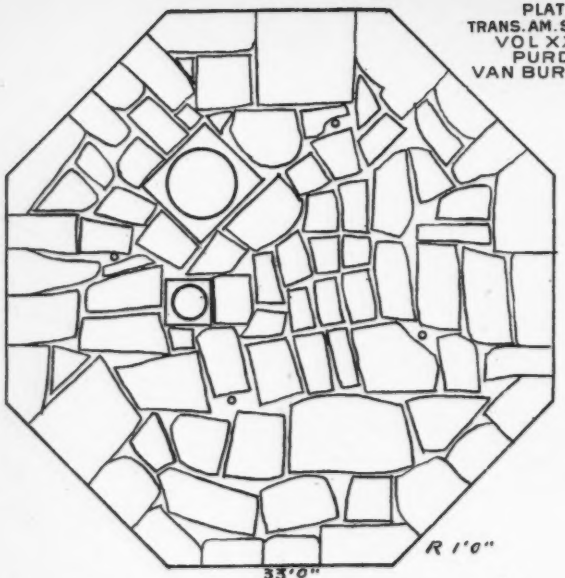
Cement 27½ bbls.

1- B

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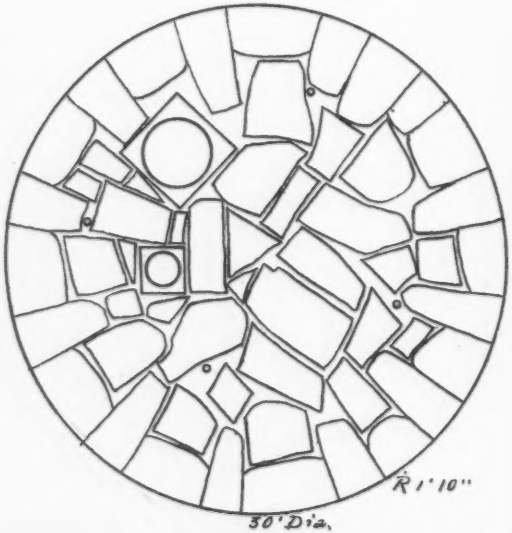
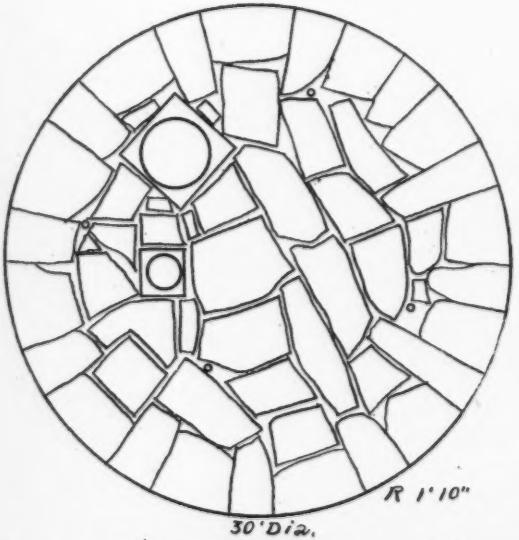
Cement 27½ bbls.

3




Commenced Aug 15th Finished Aug 16th
Cement 31 bbls.

4

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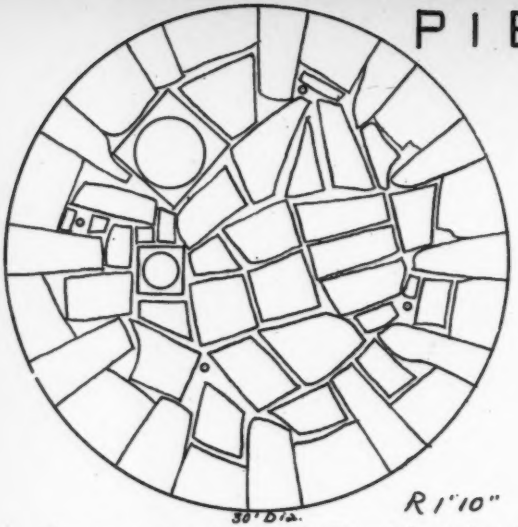
Cement 27 bbls.

# PIER No. 4.

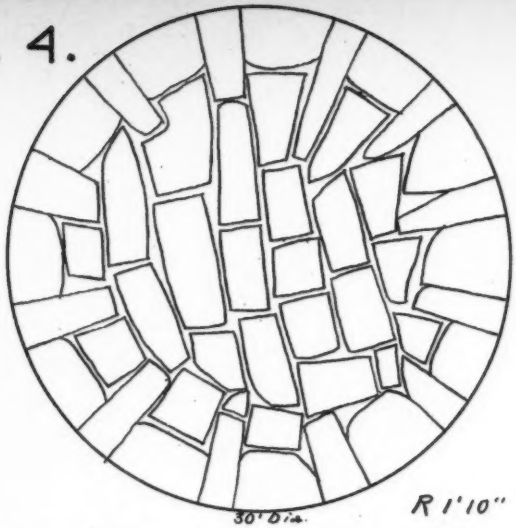
5




R 1'10"

Commenced Aug 19th Finished Aug 20th
45.7 Cu.yds. 32 bbls. Cement

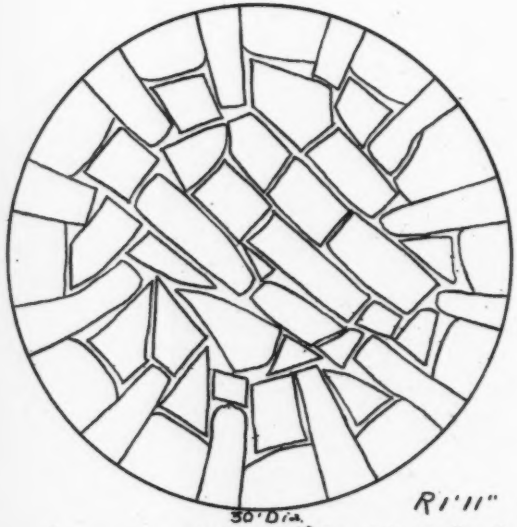
6




R 1'10"

Commenced Aug 28th Finished Aug. 29th
48.0 Cu.yds. 35 bbls. Cement

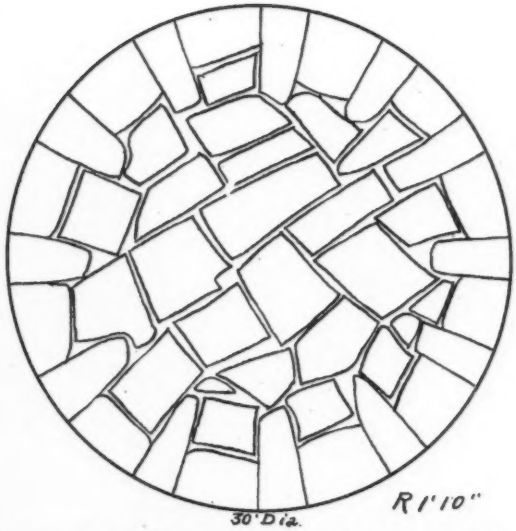
8




R 1'11"

Commenced Aug 31st Finished Sept. 1st
50.18 Cu.yds. 37 bbls. Cement

9



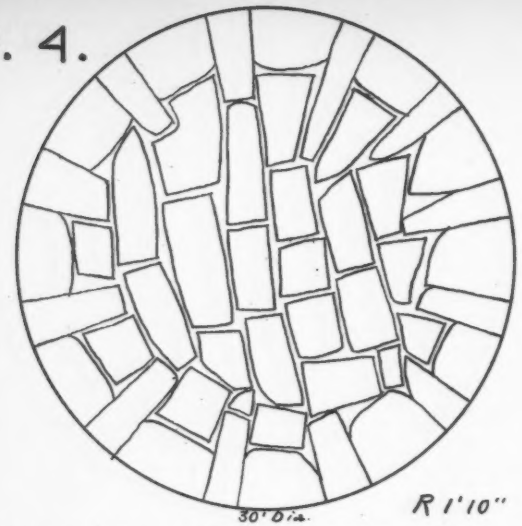

R 1'10"

Commenced Sept. 1st Finished Sept. 2nd
48.0 Cu.yds. 33 bbls. Cement

PLATE XXX
TRANS. AM. SOC. CIV. ENGRS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.

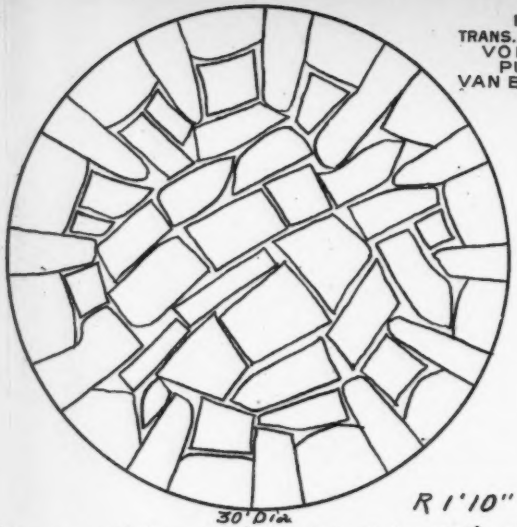
Nº. 4.

6



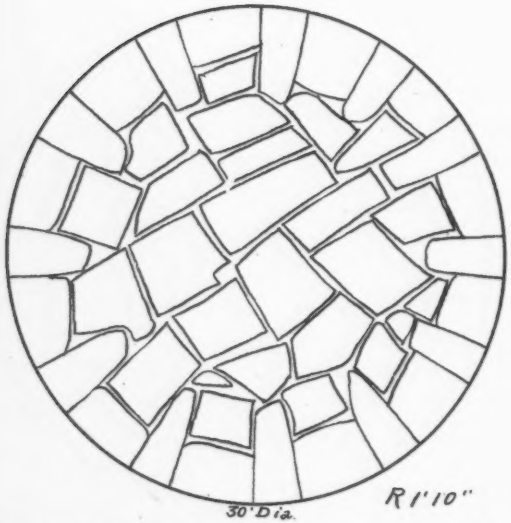

Commenced Aug 28th Finished Aug. 29th
48.0 Cu. yds. 35 bbls. Cement

7



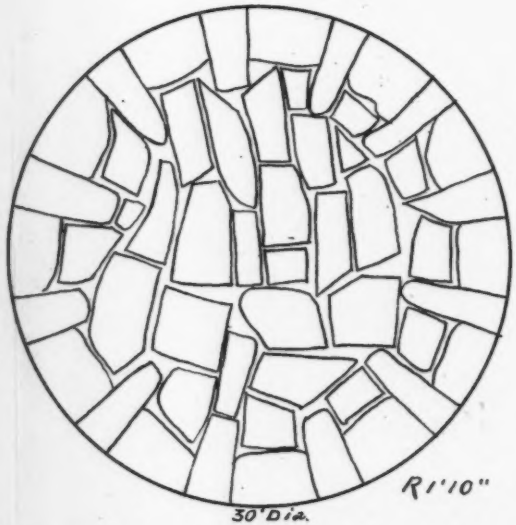

Commenced Aug. 29th Finished Aug 31st
50.18 Cu. yds. 35 bbls. Cement

9




Commenced Sept. 1st Finished Sept. 2nd
48.0 Cu. yds. 33 bbls. Cement

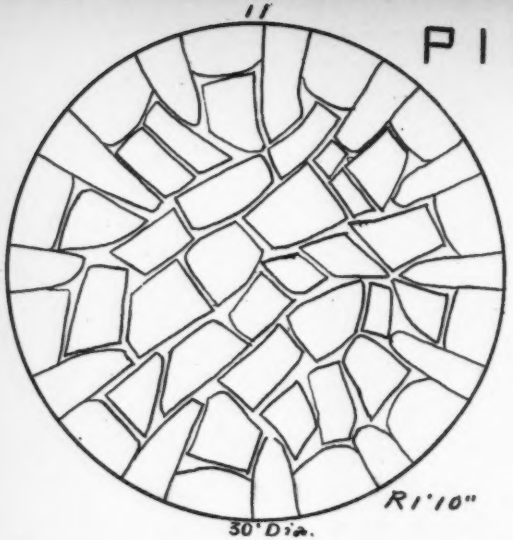
10




Commenced Sept. 2nd Finished Sept. 3rd
48 Cu. yds. 33 bbls. Cement

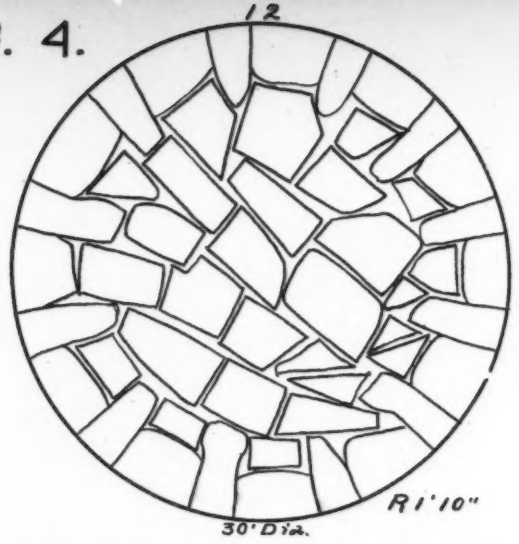
# PIER No. 4.

11



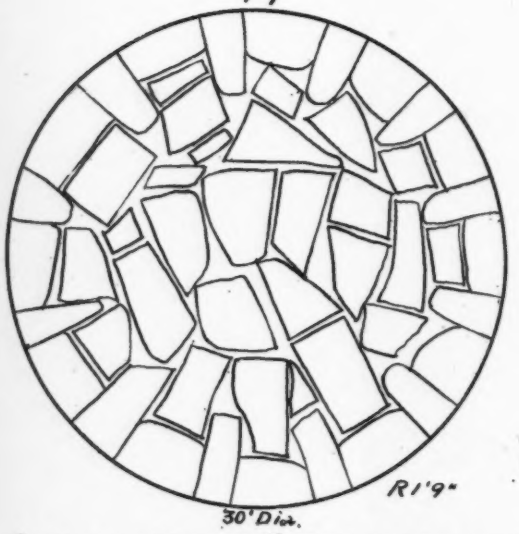

Commenced Sept. 3rd Finished Sept 4th
Cement 33 bbls.

12



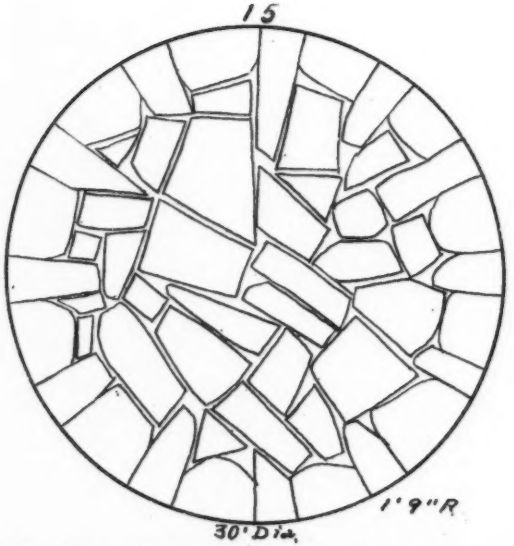

Commenced Sept. 4th Finished Sept. 6th
Cement 36½ bbls.

14




Commenced Sept. 10th Finished Sept. 17th
Cement 36 bbls.

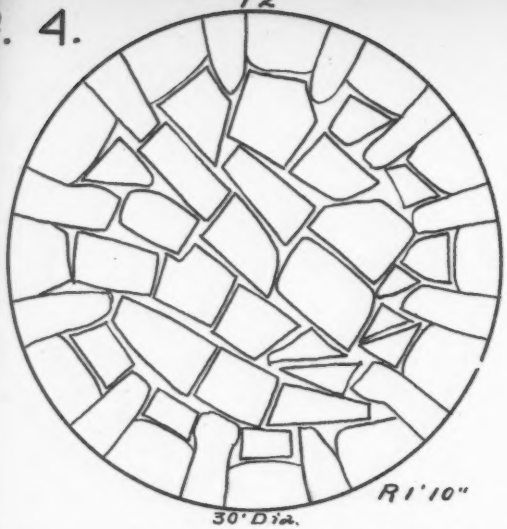
15




Commenced Sept. 17th Finished Sept. 19th
Cement 37 bbls.

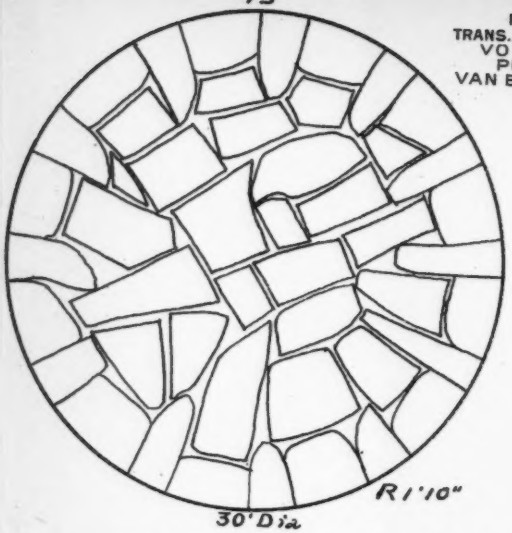
No. 4.

12



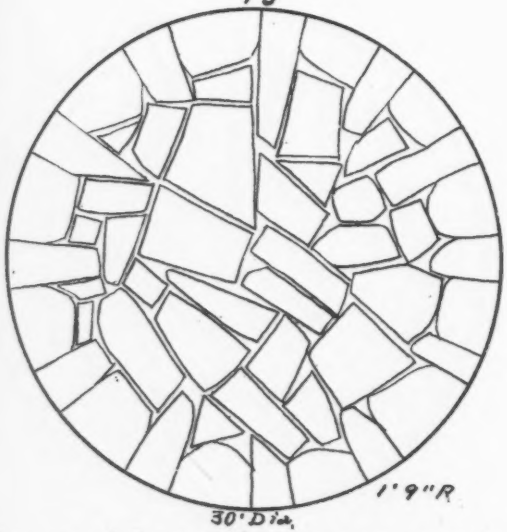

Commenced Sept. 4th Finished Sept. 6th
Cement 36½ bbls.

13



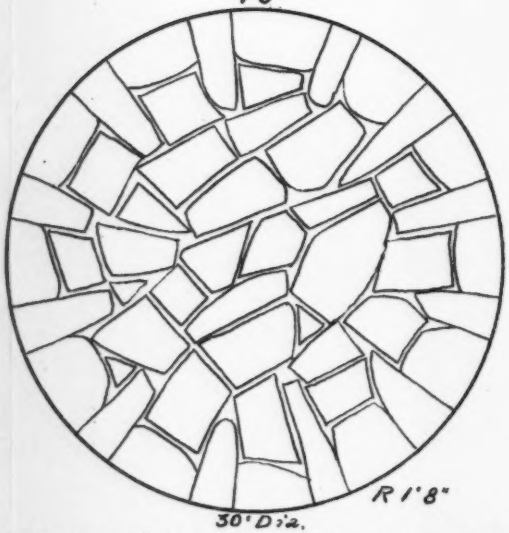

Commenced Sept. 7th Finished Sept. 8th
Cement 33 bbls.

15




Commenced Sept. 17th Finished Sept. 19th
Cement 37 bbls.

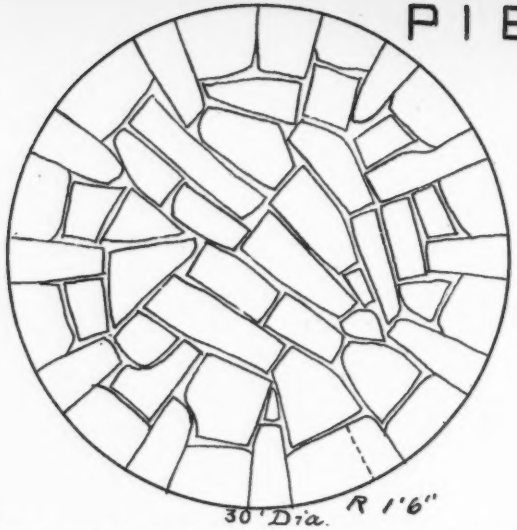
16




Commenced Sept. 19th Finished Sept. 21st
Cement 33 bbls.

# PIER No. 4.

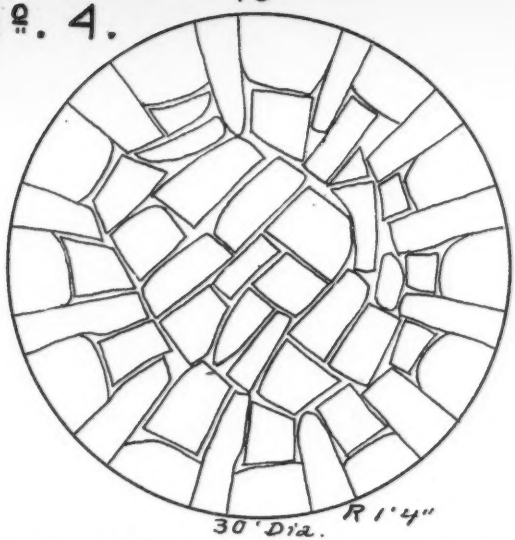
17



30' Dia. R 1'6"

Commenced Sept. 23rd Finished Sept. 24th
Cement 28 1/2 bbls.

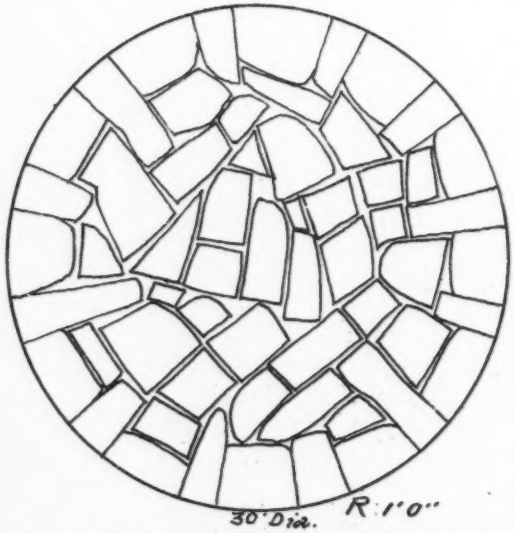
18



30' Dia. R 1'4"

Commenced Sept. 24th Finished Sept. 25th
Cement 23 bbls.

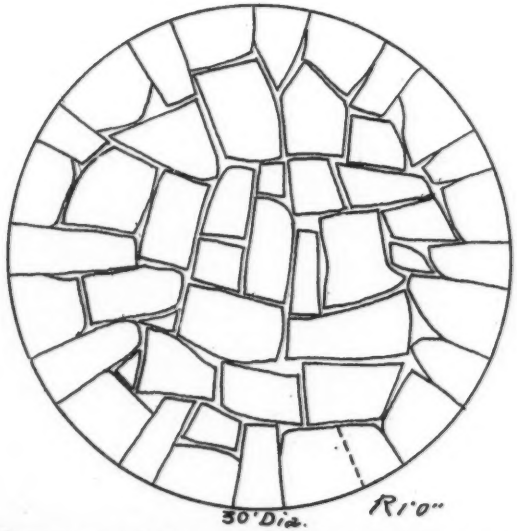
20



30' Dia. R 1'0"

Commenced Sept. 26th Finished Sept. 28th
Cement 15 bbls.

21



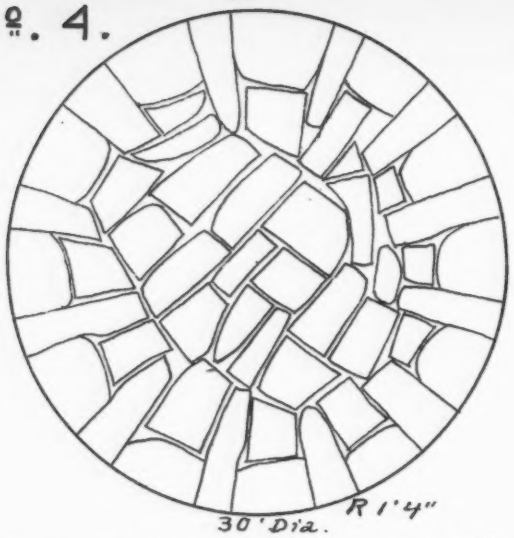
30' Dia. R 1'0"

Commenced Sept. 28th Finished Oct. 1st
Cement 14 1/2 bbls.

ER Nº. 4.

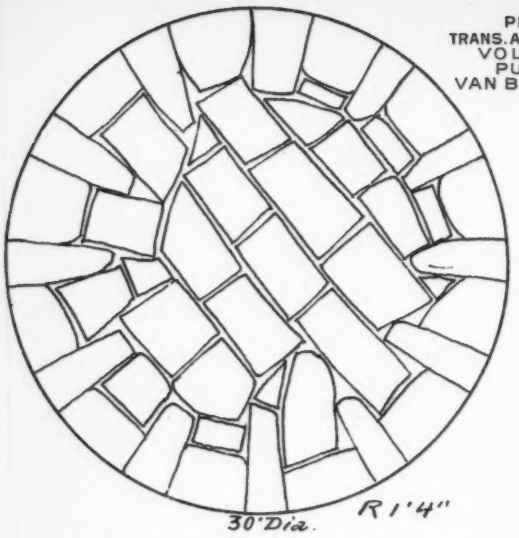
PLATE XXXII
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.

18



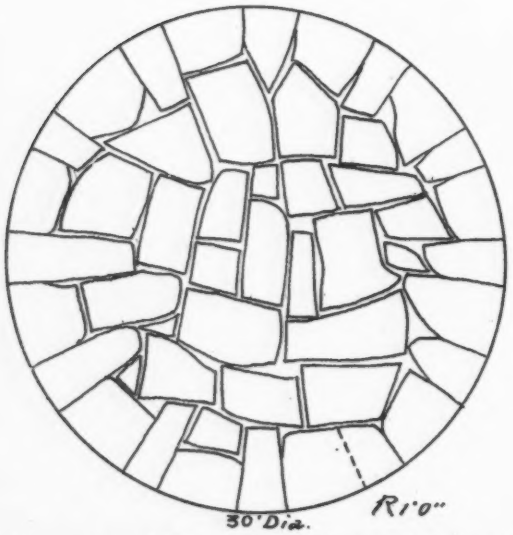

Commenced Sept. 24th Finished Sept. 25th
Cement 23 bbls.

19



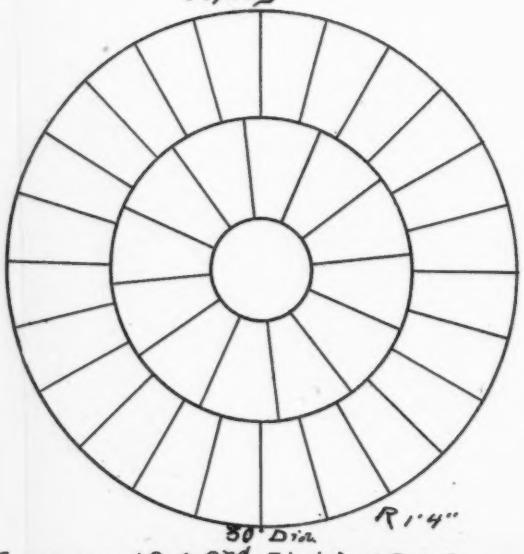

Commenced Sept. 25th Finished Sept. 26th
Cement 19½ bbls.

21

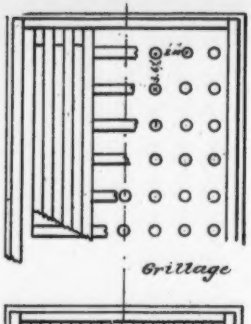



Commenced Sept. 28th Finished Oct. 1st
Cement 14½ bbls.

Coping




Commenced Oct. 2nd Finished Oct. 5
Cement 18½ bbls.




PIER 7 - AS BUILT

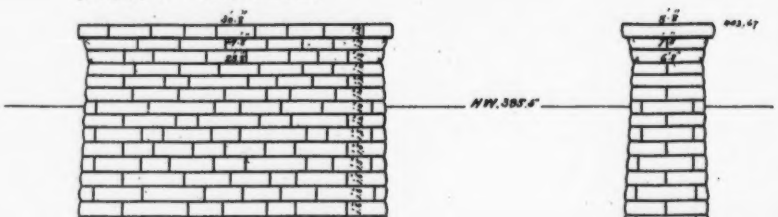
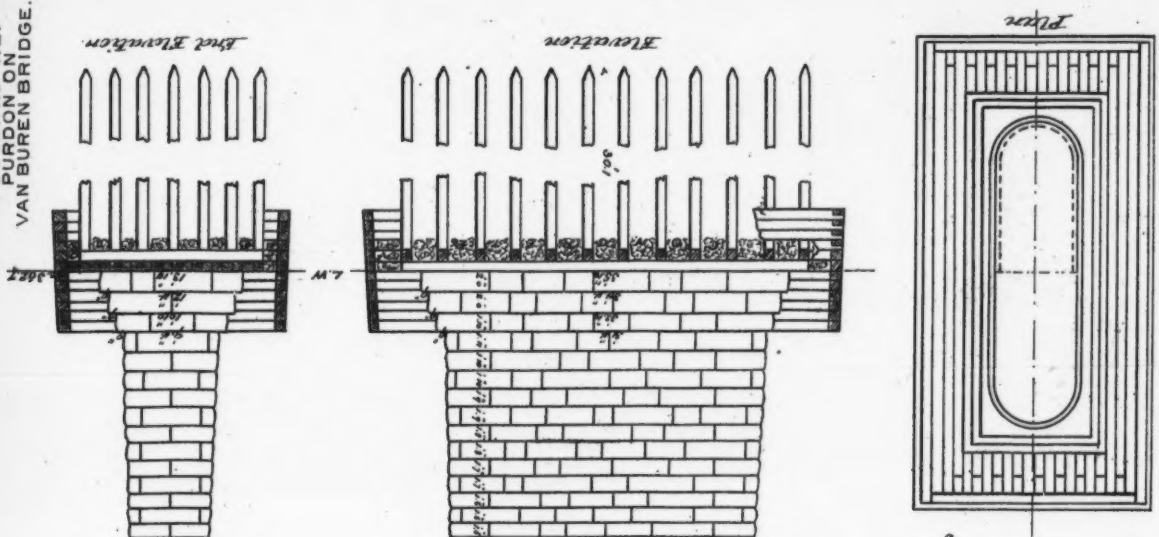


PLATE XXXIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XX. N° 412.
PURDON ON
VAN BUREN BRIDGE.



PIER 10

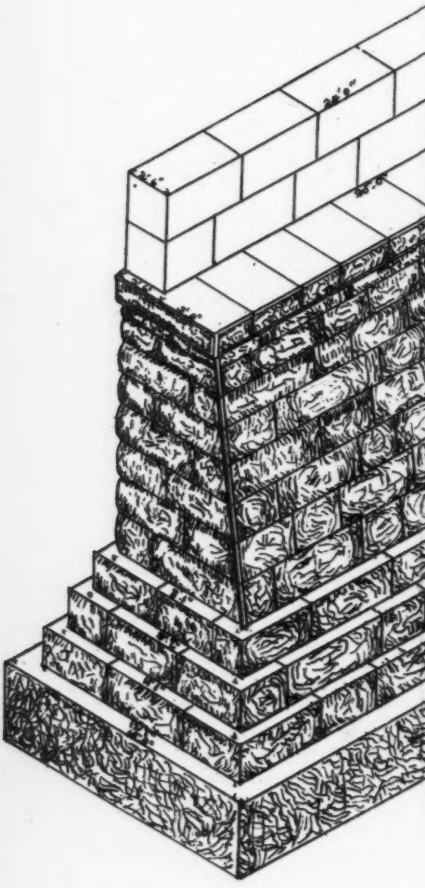
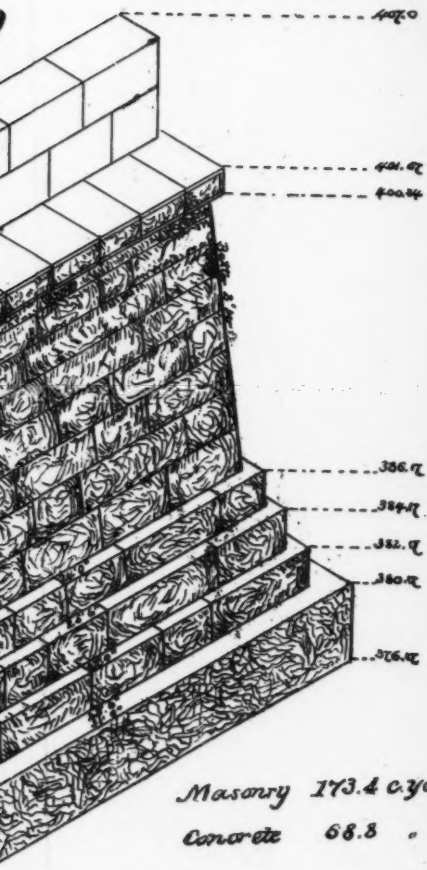


PLATE XXXIV
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.




Masonry 173.4 c.yds.
Concrete 68.8 "

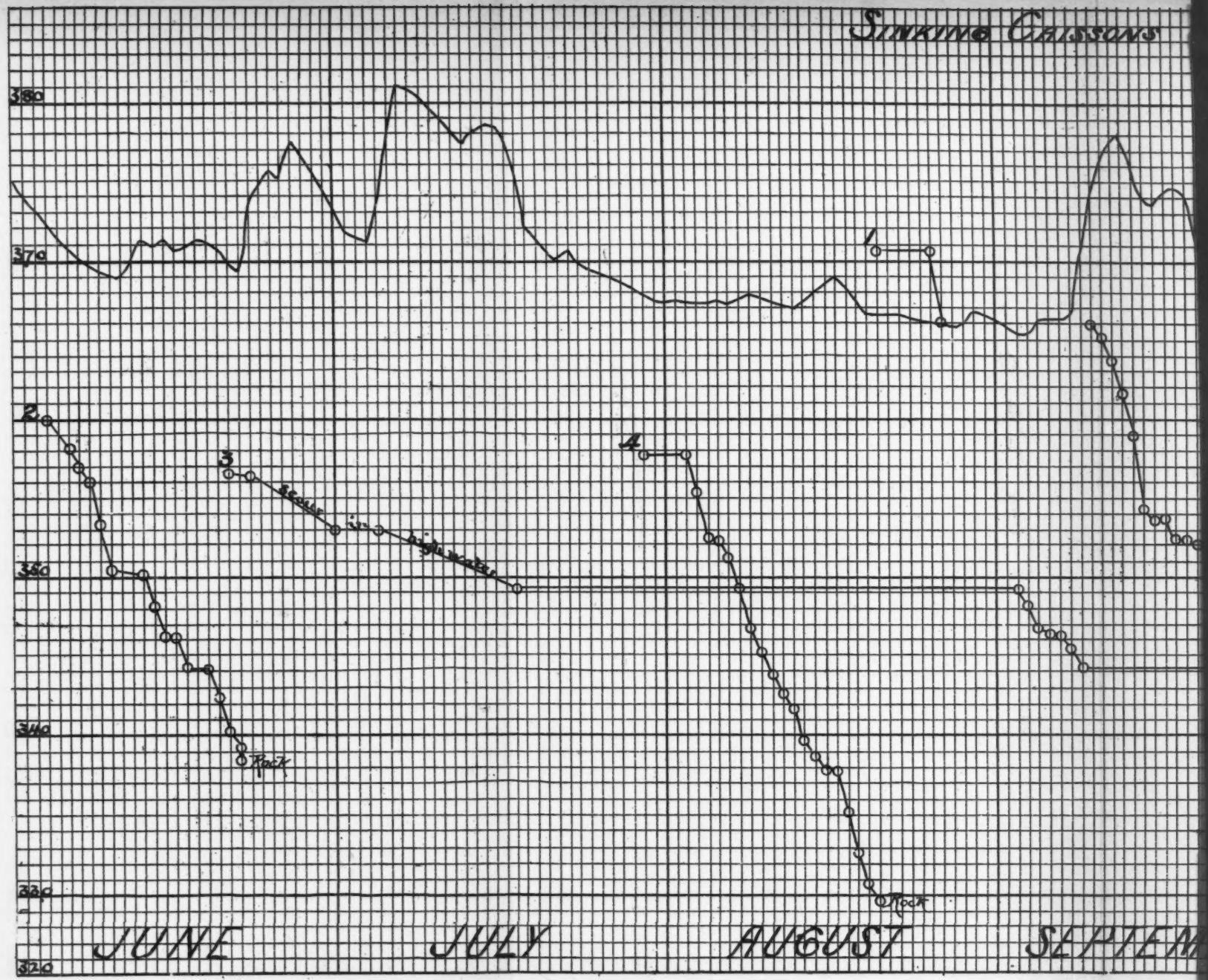
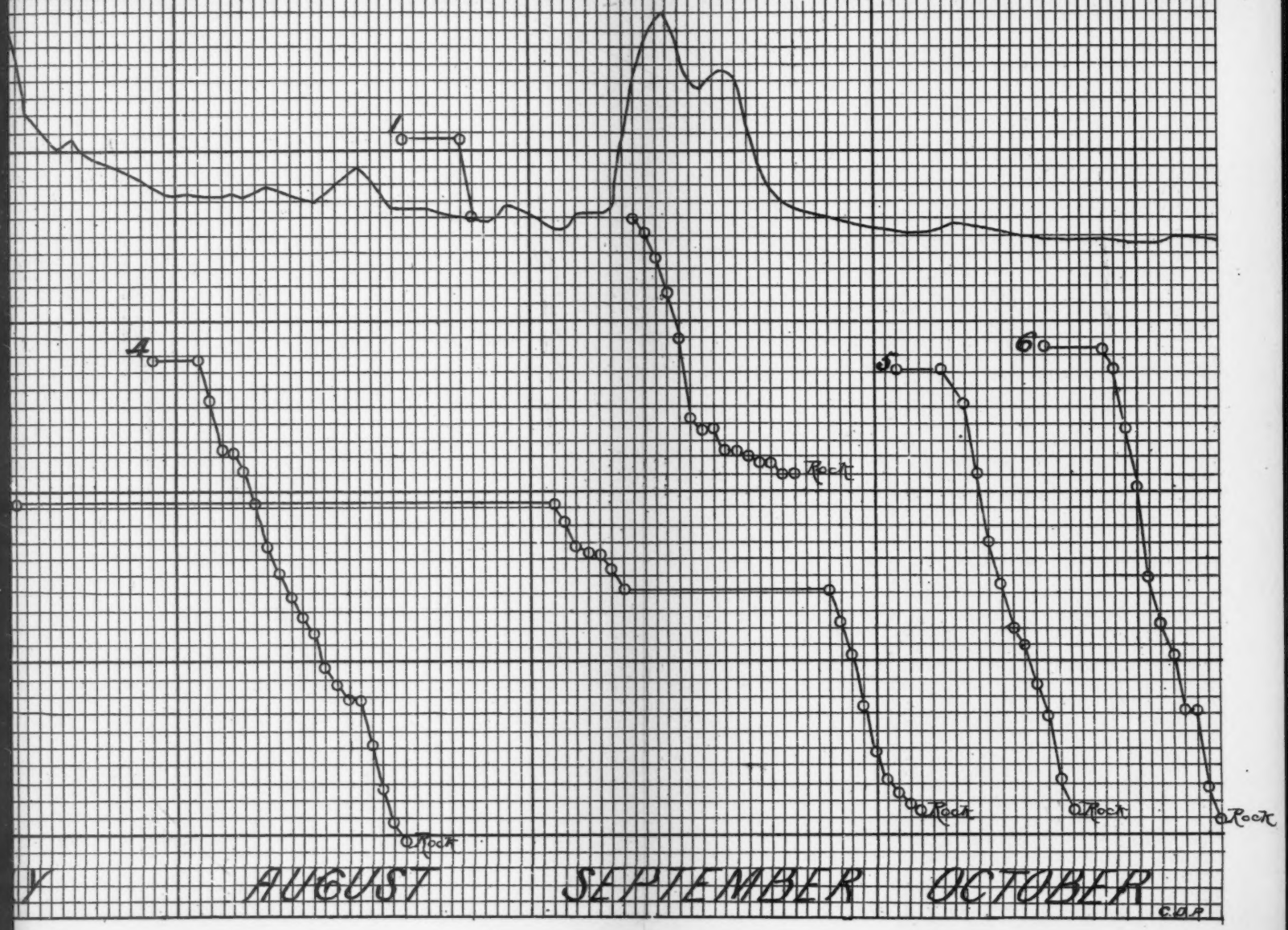
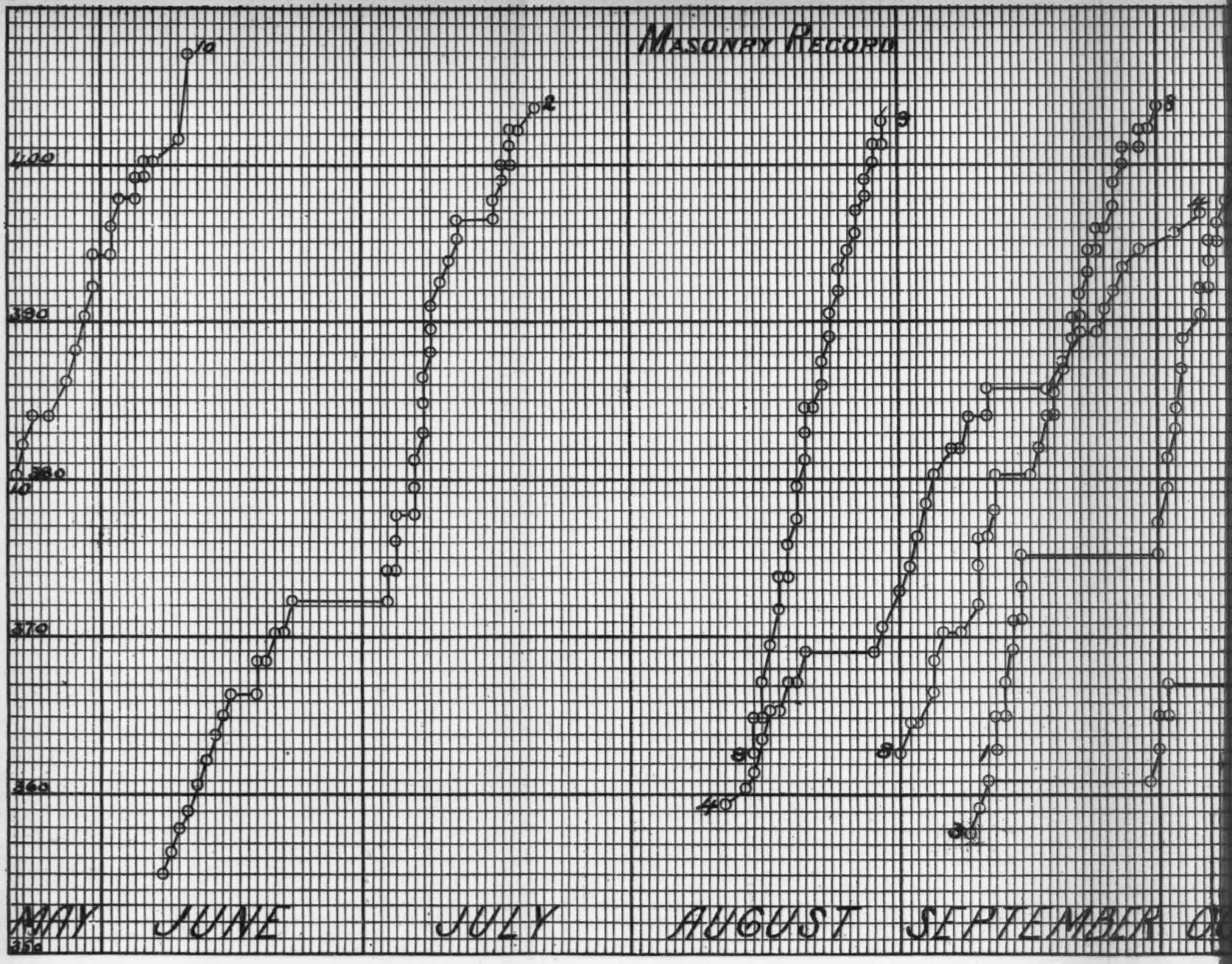

Sinking Caissons
380
370
360
350
340
330
320
1
2
3
4
Rock
Rock
JUNE
JULY
AUGUST

PLATE XXXV.
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.

# Sinking Caissons




MASONRY RECORD
400
390
380
370
360
350
MAY
JUNE
JULY
AUGUST
SEPTEMBER

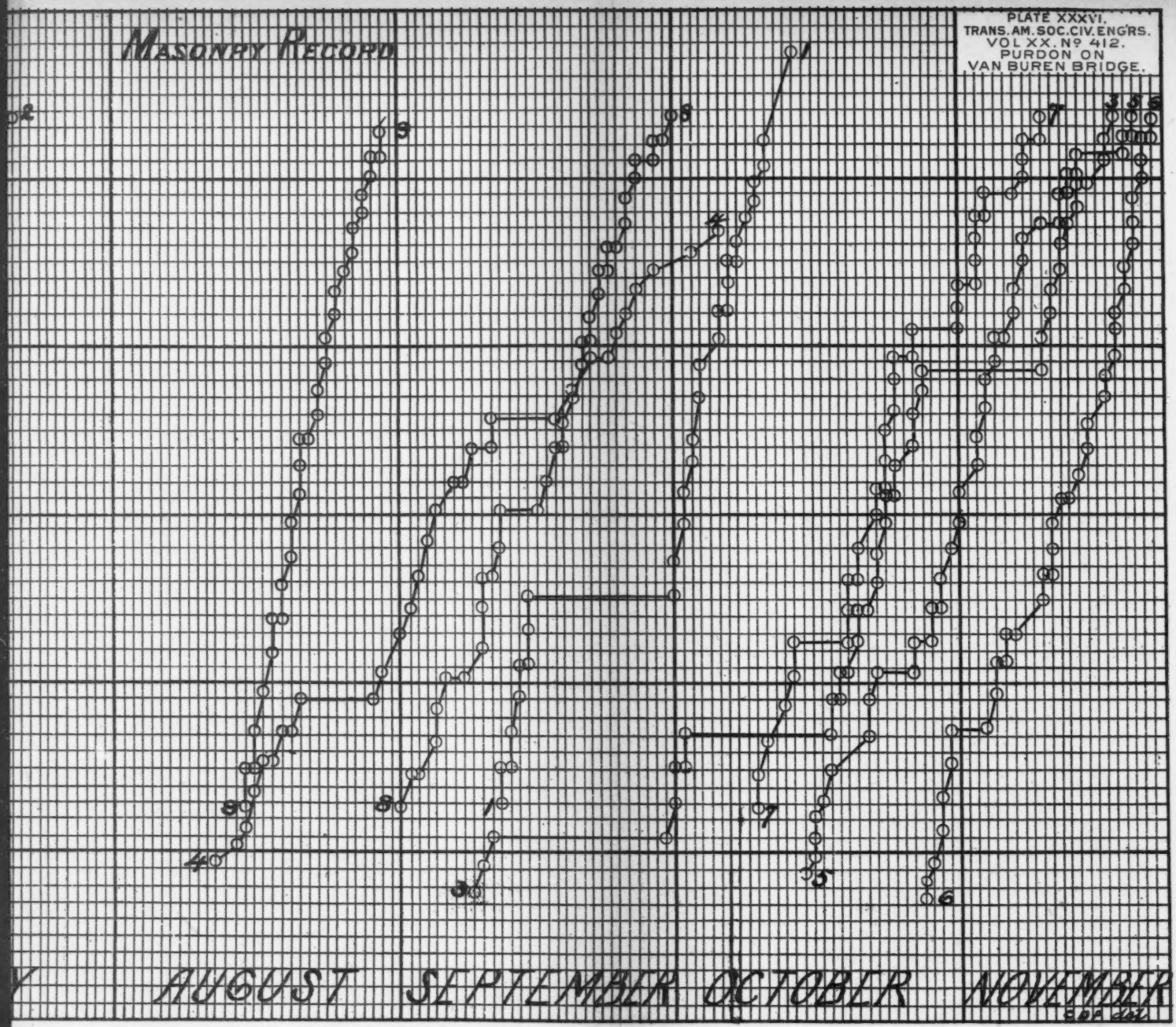


PLATE XXXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.
MASONRY RECORD
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER

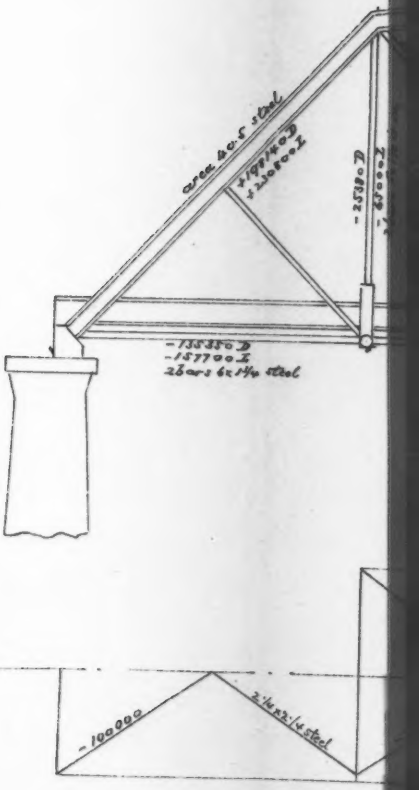
Dead load per ft 2575 lbs

Live " " " according to

Width 20 ft ctoc of trusses

Ht of posts 30, 34, 38 and 42 ft

9 panels 28 ft 1 in each



Bs

ting to specifications

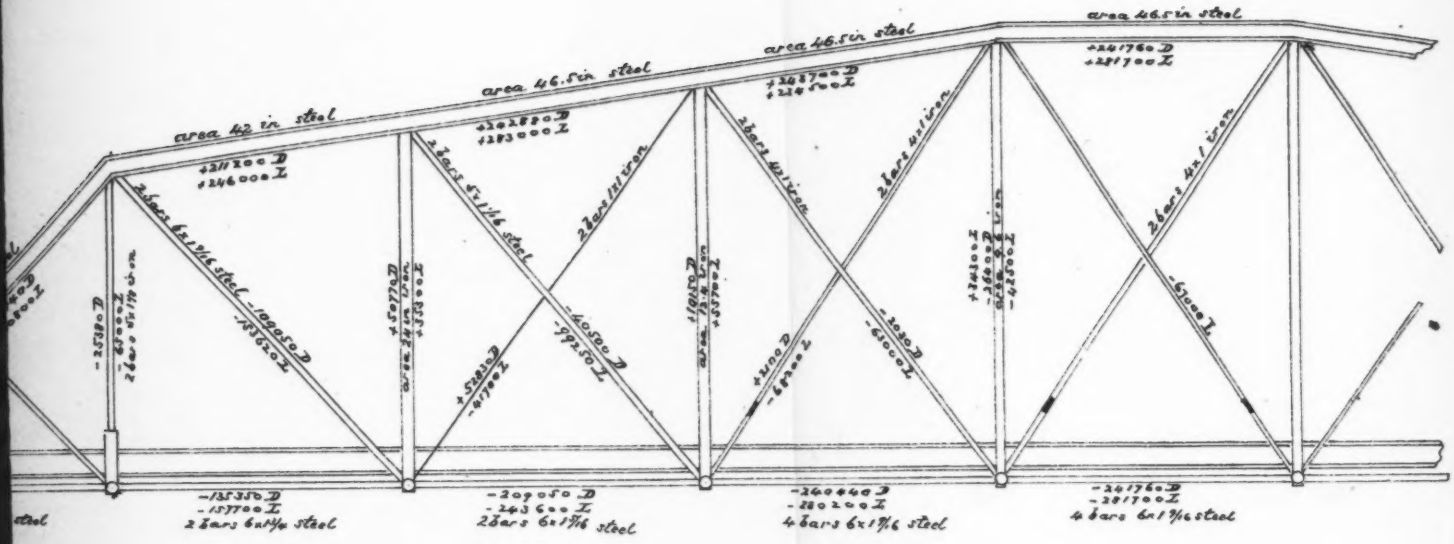
ses

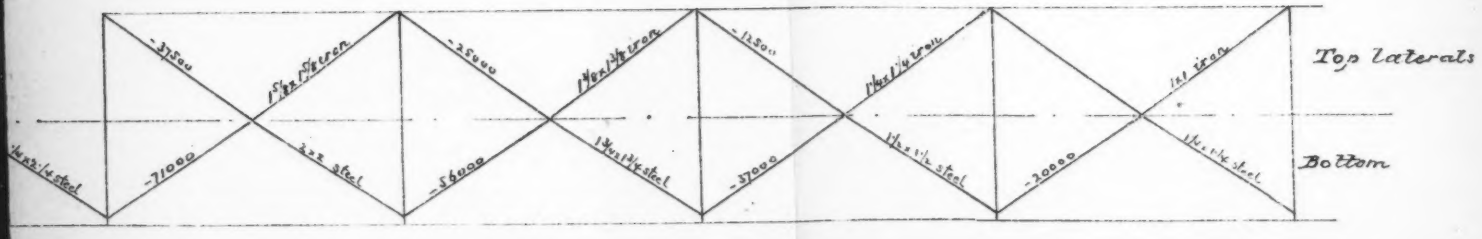
42 ft

# STL AND SF RY - VAN BUREN BRIDGE

## 3 SPANS 252 FT 9 IN c to c END PINS

PLATE XXXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE



# 4 SPANS 162 Ft ctoc END PINS

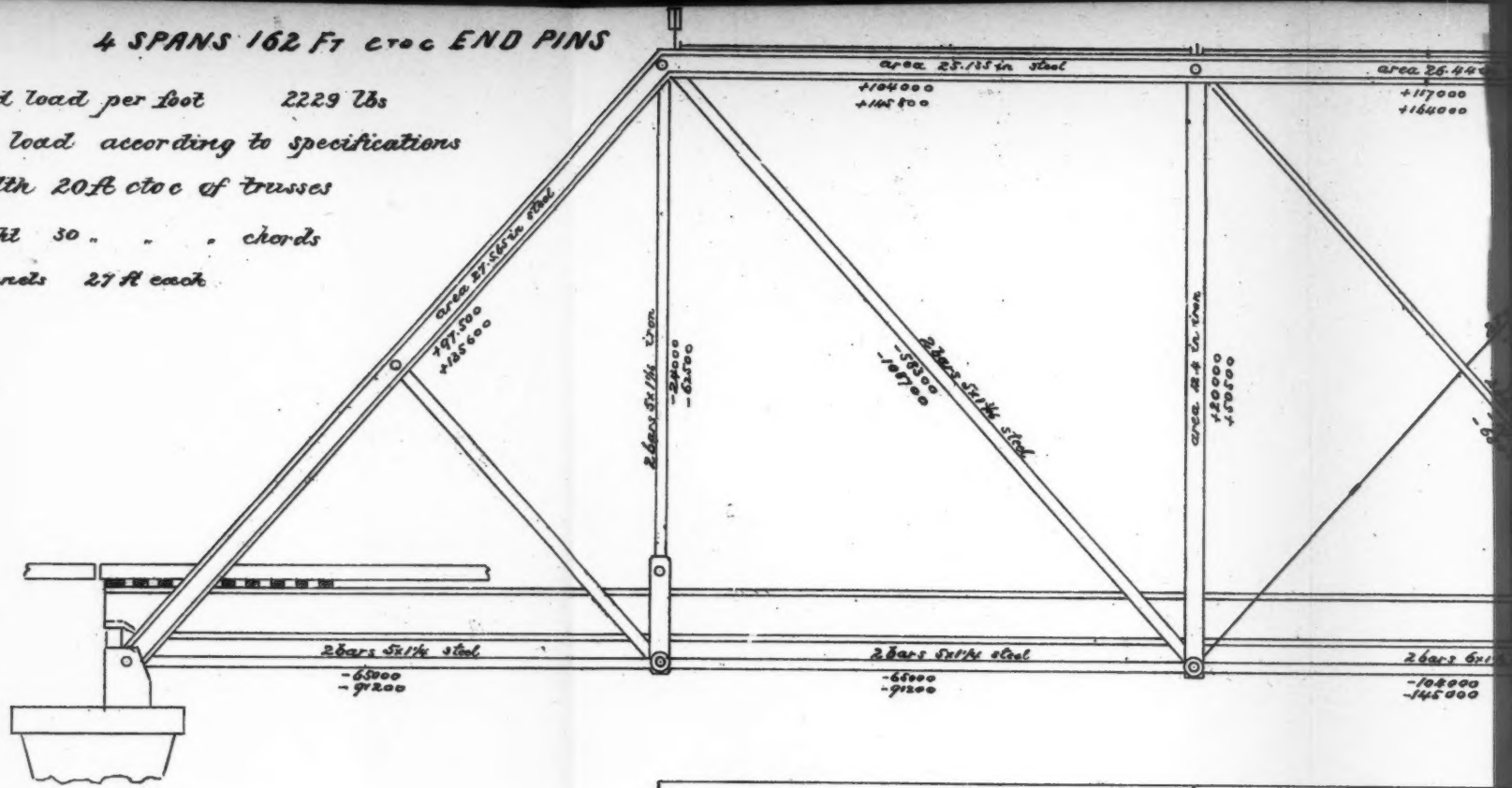
Dead load per foot 2229 lbs

Live load according to specifications

Width 20ft ctoc of trusses

Height 30 " " " chords

6 panels 27 ft each



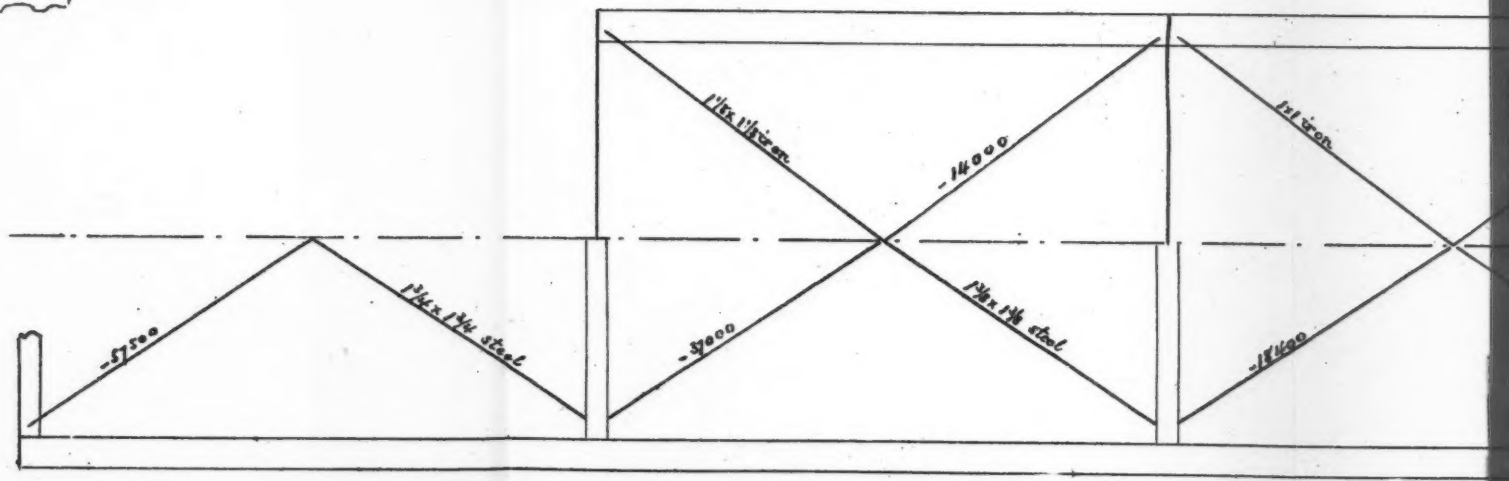
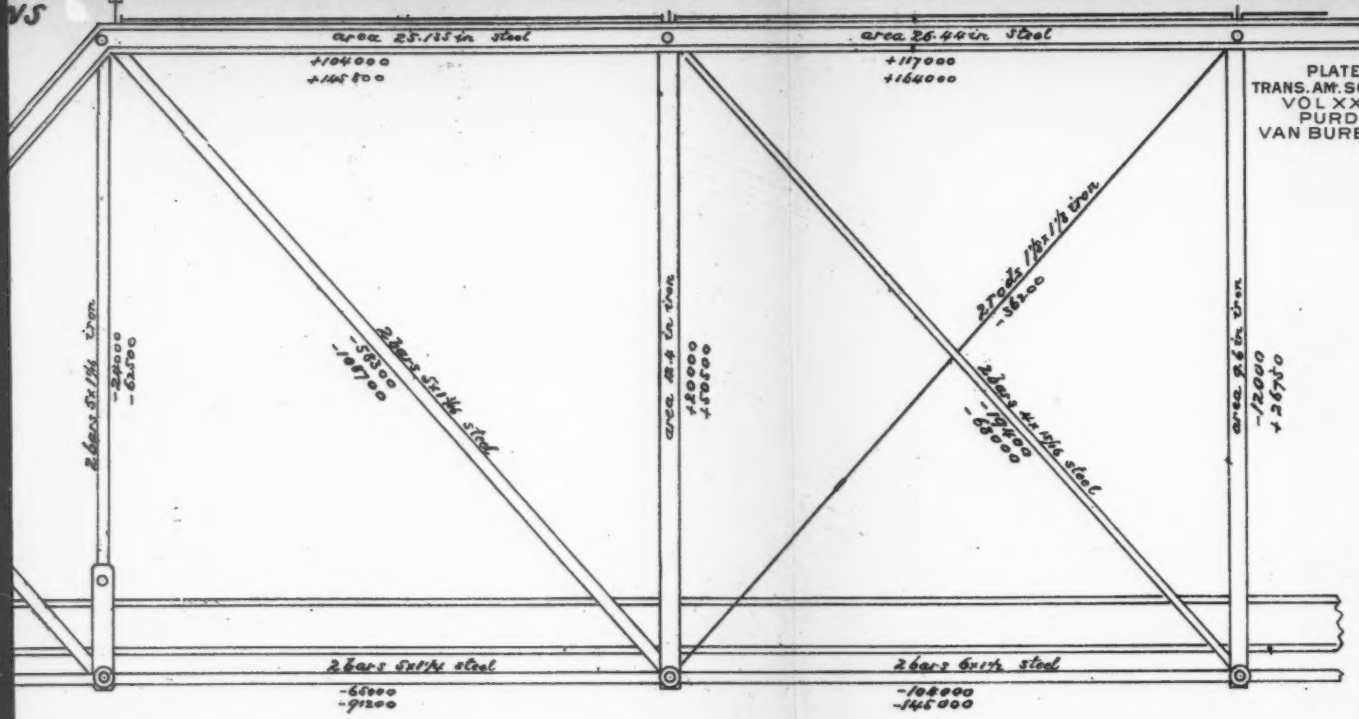
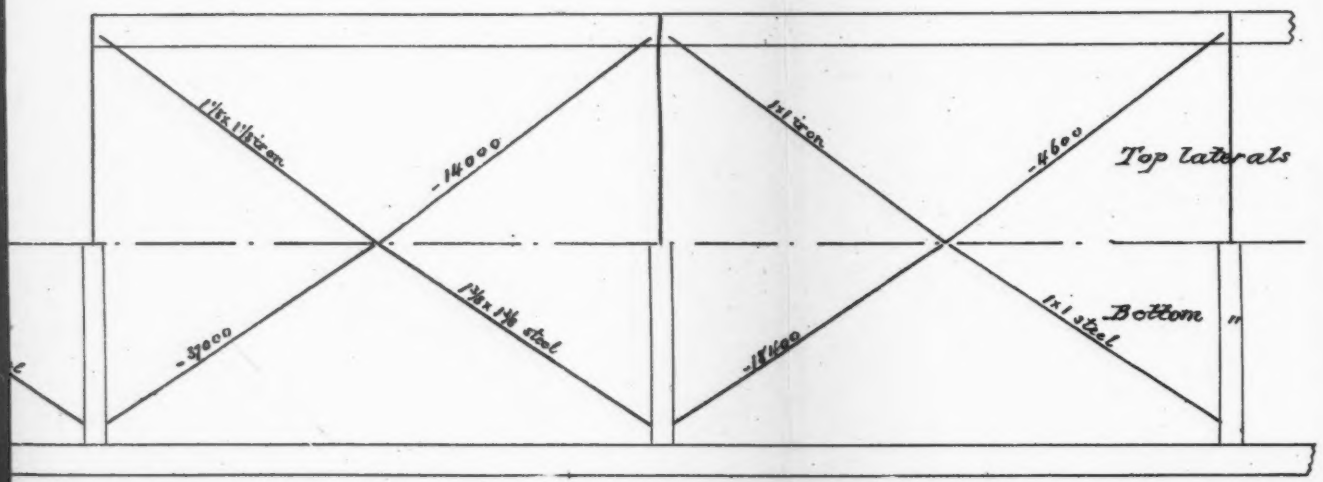



PLATE XXXVIII
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX. Nº 412.
PURDON ON
VAN BUREN BRIDGE.




C.D.P

# St L and S.F. Ry - VAN BUREN BRIDGE

## DRAW SPAN

Dead load per ft 2585 lbs

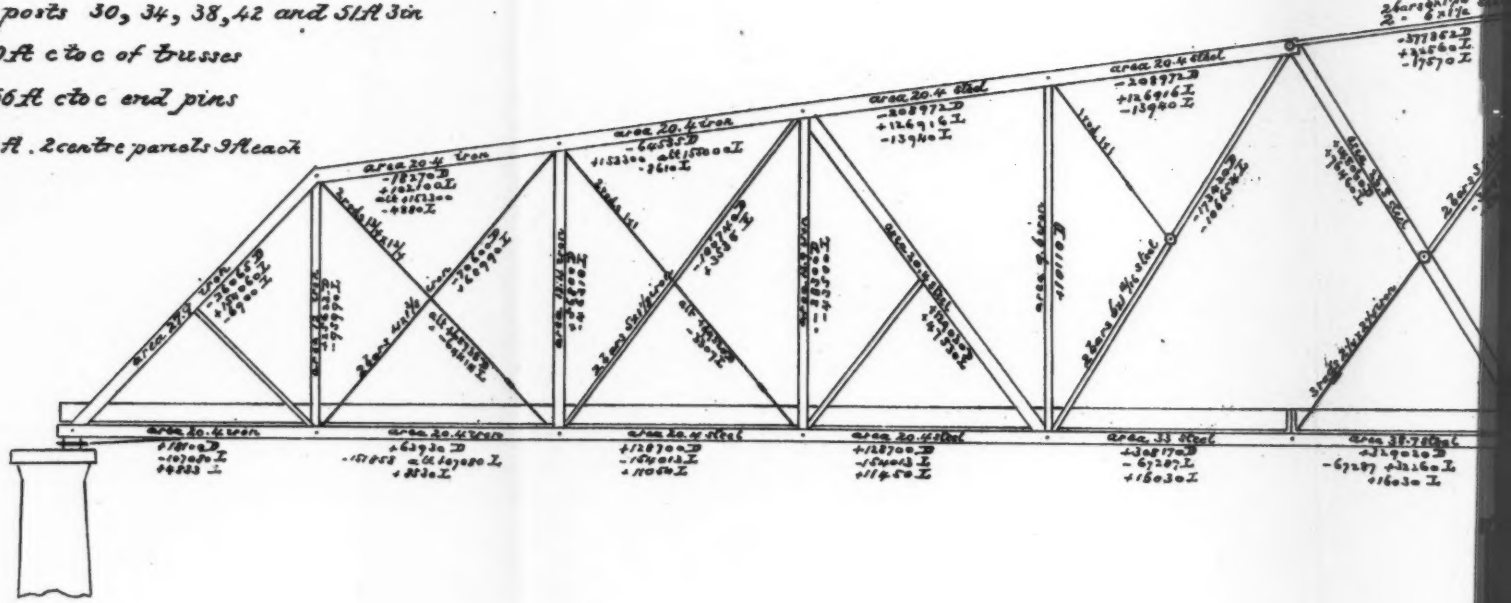
Live . . . according to specification

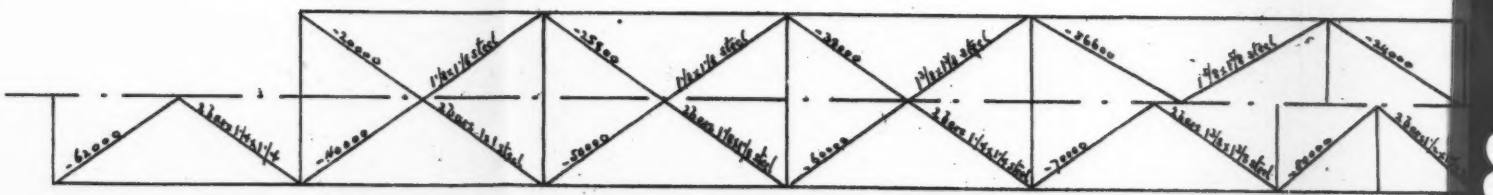
Height of posts 30, 34, 38, 42 and 51 ft 3 in

Width 20 ft c to c of trusses

Length 366 ft c to c end pins

12 panels 29 ft . 2 centre panels 9 ft each





# St L and S.F. Ry - VAN BUREN BRIDGE

## DRAW SPAN

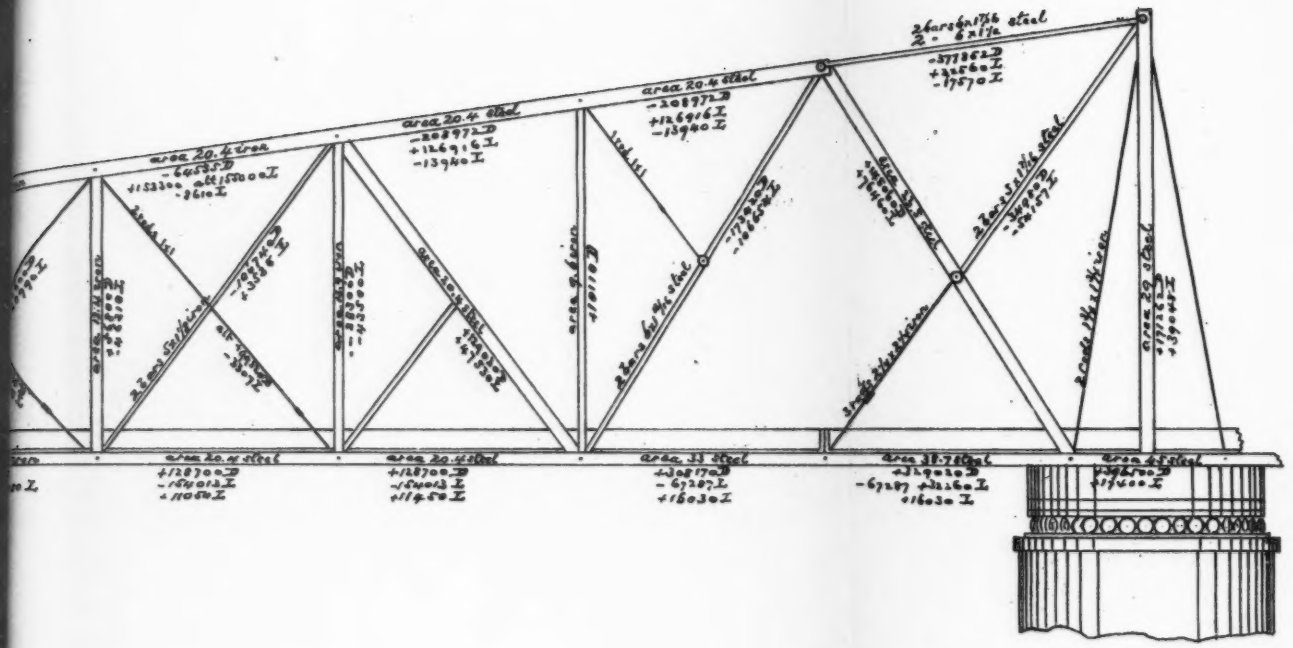
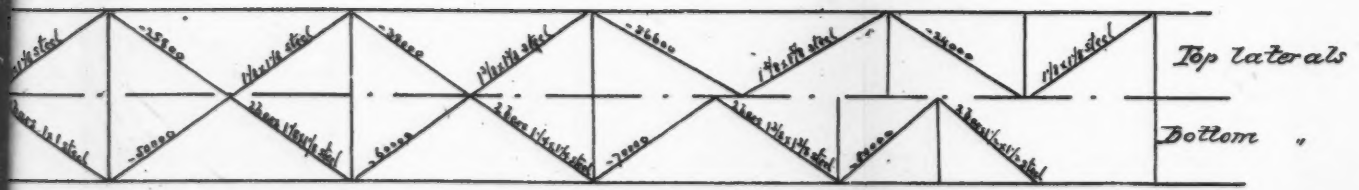



PLATE XXXIX
TRANS. AM. SOC. CIV. ENG'RS.
VOL XX: Nº 412.
PURDON ON
VAN BUREN BRIDGE.